SCHOOL OF CIVIL ENGINEERING

JOINT HIGHWAY RESEARCH PROJECT

JHRP-77-19

APPLICATION OF THE UTCS-1 NETWORK SIMULATION MODEL TO SELECT OPTIMAL SIGNAL TIMINGS IN A MULTI-LINEAR STREET SYSTEM

R. N. Carini, Jr.

PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

Interim Report

APPLICATION OF THE UTCS-1 NETWORK SIMULATION MODEL TO SELECT OPTIMAL SIGNAL TIMINGS IN A MULTI-LINEAR STREET SYSTEM

TO: J. F. McLaughlin, Director
Joint Highway Research Project

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

November 8, 1977

Project No.: C-36-66G

File: 8-7-7

The attached Interim Report titled "Application of the UTCS-1 Network Simulation Model to Select Optimal Signal Timings in a Multi-Linear Street System" is presented as partial fulfillment of the objectives of the research. It has been authored by Mr. Raymond N. Carini, Jr., Graduate Instructor in Research on our staff, under the direction of Professor K. C. Sinha.

The Report details a modification of the UTCS-1 simulation model of FHWA by the researcher and its application to a network containing three linear systems of signals in West Lafayette, Indiana. Considerable improvement in the flow of traffic as evaluated by several measures of effectiveness was shown to be possible for the streets in the system over that obtained by current signal timing plans.

The next phase of this Study will be implementation of the recommended timing plans in West Lafayette and evaluation of the benefits. The model will then be applied to one or more higher volume arterial streets in other cities as recommended by the Traffic Engineering Division of ISHC and similarly evaluated. Upon such validation a brief Guideline Manual for use of the simulation model with any network of streets will be prepared and the model will be placed in operating condition on the ISHC computer in Indianapolis.

The Report is submitted as partial fulfillment of the objectives of this Study.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Associate Director

HLM:ms

cc: W. L. Dolch
R. L. Eskew
G. D. Gibson
W. H. Goetz
M. J. Gutzwiller
G. K. Hallock
D. E. Hancher
K. R. Hoover
G. A. Leonards
R. F. Marsh
R. D. Miles
P. L. Owens
G. T. Satterly
C. F. Scholer
M. B. Scott
K. C. Sinha
C. A. Venable
L. E. Wood
E. J. Yoder
S. R. Yoder

Interim Report

APPLICATION OF THE UTCS-1 NETWORK SIMULATION MODEL TO SELECT OPTIMAL SIGNAL TIMINGS IN A MULTI-LINEAR STREET SYSTEM

by

Raymond N. Carini, Jr.
Graduate Instructor in Research

Joint Highway Research Project

Project No.: C-36-66G

File No.: 8-7-7

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project
Engineering Experiment Station
Purdue University

in cooperation with the

Indiana State Highway Commission

Purdue University
West Lafayette, Indiana
November 8, 1977

ACKNOWLEDGEMENTS

The author is sincerely appreciative of the timely advice, guidance and understanding extended to him by his major professor, Dr. Kumares C. Sinha.

Professor Harold L. Michael and Dr. Virgil Anderson provided much needed assistance and suggestions; the author is very grateful for this.

Gratitude is also extended to the Joint Highway Research Project for the financial assistance provided for this study as well as the support for furthering his education as an engineer.

The writer also thanks his colleagues with whom many helpful and thought-provoking discussions were held. In addition, Dana Hall assisted in the initial stages of the project; Pat Schur provided expert assistance with the statistics involved in this research. Thanks are also due to Mrs. Marian Sipes for her cheerfulness in typing the manuscript.

Special thanks go to my wife, Rose, for her loving support and encouragement throughout the course of this endeavor.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF TABLES (Continued)

LIST OF FIGURES

LIST OF FIGURES (Continued)

ABSTRACT

Carini, Jr., Raymond Nicholas. M.S.C.E., Purdue University, December 1977. Application of the UTCS-1 Network Simulation Model to Select Optimal Signal Timings in a Multi-Linear Street System. Major Professor: Kumares C. Sinha.

The objective of this study was to demonstrate and apply a computer model capable of simulating automobile traffic on an urban roadway. With the many urban intersections controlled by traffic signals, this investigation sought ways to improve the flow of traffic through signalized systems. Transportation System Management guidelines were recognized and emphasis was applied accordingly. The UTCS-1 network flow simulation model for urban traffic was used in this research. Computer simulation provided detailed, quantitative output for a good insight into a particular alternative's merits in relation to others. Emphasis was placed on retiming signals to provide for the better flow of traffic. Measures of effectiveness used in comparing alternatives included network average speed, carbon monoxide emissions, fuel efficiency, delay, and stops per vehicle-trip. Additionally, major and minor street delay was considered.

An effort was made to further substantiate the model's validity, which has been widely used elsewhere for analyzing traffic engineering improvements. Relationships were developed relating fuel efficiency and carbon monoxide emissions. The effects of cycle lengths on network operation were also studied and analyzed.

In the study of selecting an efficient signal control strategy for a study area, trade-off analyses were conducted. The traffic engineer can then more clearly see the effect that an improvement of one measure of effectiveness would have on another.

Signal timing methodologies were presented and reviewed. These methods were employed in the generation of signal timing plans. Appropriate UTCS output permits refinement of these plans based on a link by link analysis.

To encourage further usage of the UTCS-1 simulation model, the author prepared a detailed manual specifying input data needs and format. Also an exemplary simulation run was conducted to facilitate understanding of the data encoding process.

CHAPTER 1: INTRODUCTION

With the advent of the energy crisis and the subsequent concern regarding the nation's energy supplies, traffic engineers have increased their efforts to improve the flow of traffic, especially in urban areas. At the same time, the harmful effects of automobile emissions have caused alarm and steps are being undertaken to curb the pollution problem. Decreasing energy usage, pollution levels, and delay in a given area should be criteria for many traffic engineering improvements. Few would argue their importance.

By monitoring the flow of traffic over a specific region, one can predict the amount of delay experienced, pollutants emitted, and fuel consumed by automobiles. Computer simulation makes it possible to easily monitor the flow of traffic and the resultant measures of effectiveness (MOE's) such as: average speed, stops per vehicle,miles per gallon of fuel, hydrocarbons emitted, and delay per vehicle.

A computer simulation model developed for the Federal Highway Administration (FHWA) was designed specifically to determine MOE's of traffic flow on an urban street network. By altering control strategies along the roadway, the engineer can better evaluate the effectiveness of new signal timings, lane additions, turn prohibitions, etc. As such, this report will serve as a demonstration of the use of this simulation model - Urban Traffic Control System (UTCS-1S). Guidelines for the use of UTCS as well as detailed input and output data are presented in Appendix A.

Increased automobile volume on already congested urban streets necessitates better use of these existing streets. Because of the many financial constraints, the traffic engineer must use the available resources as efficiently and effectively as possible. Probably the most cost effective improvement made by a traffic engineer to a street network is one of properly timing traffic signals to improve the quality of traffic flow. In order to best select the alternative among the many signal timing options to improve flow, this project employed UTCS-1S to determine the performance of each alternative in a cost effective framework. Thus laboratory techniques can be used to help the engineer choose among alternatives generated by use of traffic engineering principles (1). Data from an actual case study was used to determine the optimal flow conditions for a local signalized street network. Because much of the automobile congestion occurs in metropolitan areas, this project specifically addressed street systems in urban areas.

Delay, pollution, and fuel consumption comparisons were dealt with simultaneously in this report. A number of MOE's were monitored to choose the optimal signal timings and offsets in a multi-linear street system. A systems approach was undertaken to examine the tradeoffs of various effectiveness measures associated with each particular operation scheme. Oftentimes a reduction of, say, delay will not be accompanied by a reduction in fuel consumption (2). Throughout the course of this research four major objectives were sought:

1. devise an optimal traffic operation scheme by specifying traffic signal phasing and timing for the study area for the morning, evening, and off peak periods;
2. evaluate the benefits resulting from adoption of an efficiently controlled network including changes in travel time, average vehicle speed, average number of stops per vehicle, and pollution levels;
3. recommend the optimal operation scheme for the test area for implementation by the proper officials;
4. develop a simplified user's manual for employing UTCS-1S in selecting optimal signal timings.

Background and Significance of Work

To attain better traffic flow that would result in less pollution, fuel consumption and delay, roads could be widened and interchanges constructed; however, any improvement project is limited by severe financial constraints. Consequently, every effort should be made to improve the efficiency of the system by use of cost effective traffic engineering methods before any costly construction is deemed necessary. Not only must the traffic improvements be beneficial with a minimum of cost but the method of evaluating the many design options must also be done in an economical manner. Often it is too costly, time consuming, or otherwise infeasible to conduct large scale experiments for each proposed alternative. Thus preliminary planning processes have been somewhat limited in evaluating specific proposed improvement measures before actually implementing them.

The use of a properly validated computer simulation package can make possible the testing of the effectiveness of various alternatives before a particular option is implemented. The simulation package, UTCS-1 (3), is capable of simulating traffic and can be used to compare flow characteristics of present volumes and existing roadway configurations with proposed changes that would affect traffic flow: turn prohibitions, signal settings, one-way streets. Analysis of critical intersections under saturated and oversaturated conditions is also incorporated into UTCS-1.

In this project the data for the case study was collected from the City of West Lafayette, Indiana and the results obtained were evaluated for selection of the best alternative of signal operation. Three arterial streets in West Lafayette, namely, State Street, Grant Street, and Northwestern Avenue were used to develop, test, and evaluate alternative signal operations. See Figure 1.

Of major significance in this project is the low cost aspect of the proposed improvements. Transportation System Management (TSM) elements are thus realized by increasing a system's efficiency with little or no cost. The short range goals of TSM are effected since

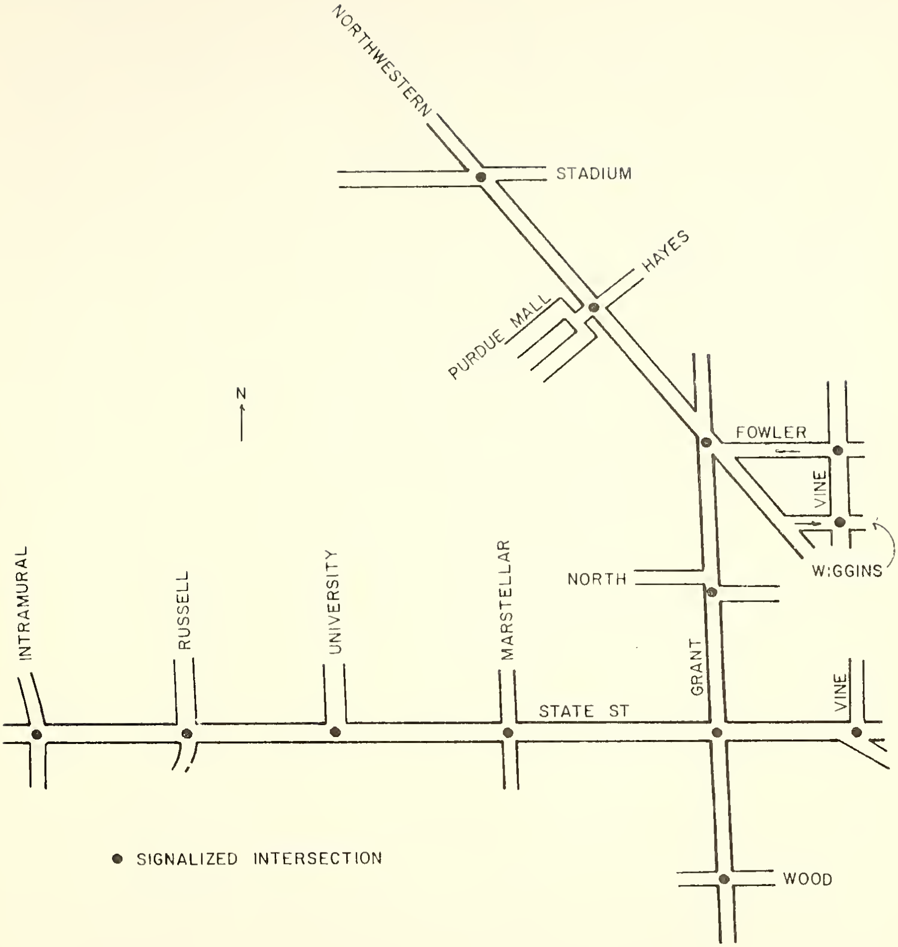

FIGURE 1
SCHEMATIC DIAGRAM OF THE TEST NETWORK

updating signal timing and deriving benefits from the improvement can be implemented in a short period of time.

While many methods have been advanced as to the best way to operate signals in congested areas, the signal timing problem is generally location-specific; unique problems often arise which vary from intersection to intersection and developed methodology does not cover these unique problems. The engineer must develop alternatives using traffic engineering principles and then analyze these proposals either in the field or by using a computer simulation program. Invariably, simulation offers much more flexibility and opportunity for consideration and analysis of more options. Once familiarity with the UTCS program is acquired traffic engineering improvement alternatives can be easily analyzed in the laboratory; a more efficient use of resources is realized by avoiding a trial-and-error system of improvement implementation. More general attributes of simulation models are as follows (4):

1. Simulation permits analysis of the interaction of the many MOE's that are to be monitored.
2. Simulation of a complex system such as traffic flow provides insight into the factors that most significantly affect its performance.
3. Experimentation with new concepts, policies, and decisions is effected economically without allocating resources for actual implementation.
4. Simulation is fast and simulates real time in less time than actual field studies.
5. Simulation can test specific problems that are unique to a particular road network and have not been studied elsewhere.

The interaction among the many MOE's, most notably, delay, exhaust emissions, and fuel consumption were given special attention in this report. Courage and Parapar (5) noted that the signal cycle length that minimizes vehicle delay at a particular intersection does not minimize the fuel consumption of vehicles traveling through the intersection. See Figure 2. Patterson (6) showed that longer cycle lengths are required to minimize emissions while simultaneously increasing

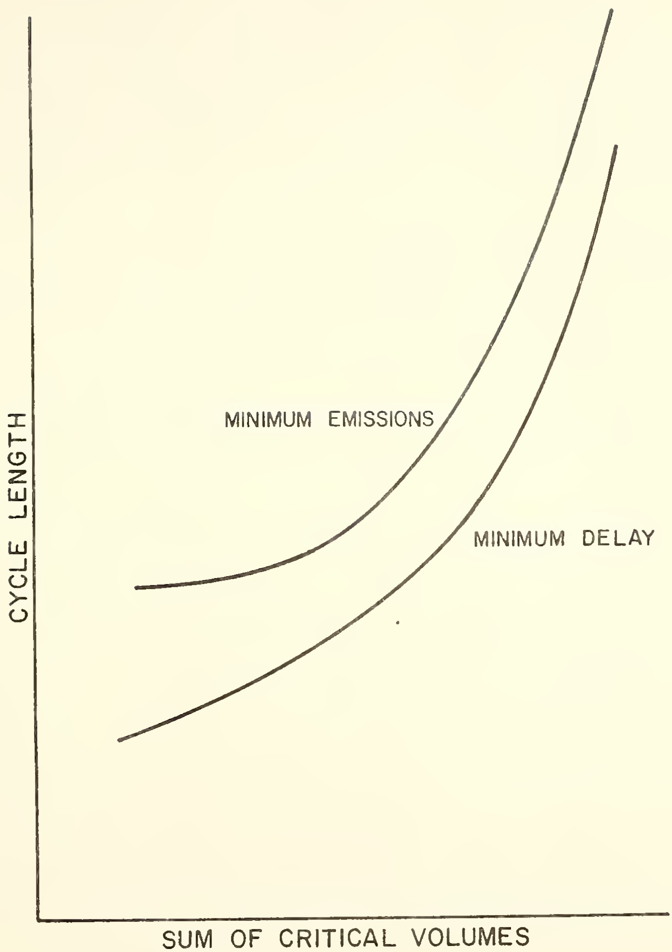

FIGURE 2: EFFECT OF CYCLE LENGTH ON DELAY AND EMISSIONS
(SOURCE: REFERENCE 6)

overall delay experienced by vehicles traveling through the intersection. He emphasizes the need for consideration of the interaction of traditional traffic engineering and motor vehicle emissions: "Situations come up in which the goals of traffic flow and air pollution control engineers are at variance" (7). See Figure 3. Figures 2 and 3 were developed by researchers investigating specific volume conditions at single isolated signalized intersections. While these observations were made under certain conditions, dissimilar results were found when considering a system of signals along an arterial as studied in this report.

Fuel consumption for a given traffic stream will vary depending on many factors including vehicle weight, engine size, model year, and overall condition. Of interest in this study was the fuel consumed because of unnecessary slow speeds and vehicle delay. The frequency of speed changes was also considered. By better signal timings, one can effectively reduce these adverse effects to achieve a more efficient system.

In addition to the consideration of fuel consumption was the regard for exhaust emissions. Deterioration of the urban environment is largely attributed to automobile congestion and the resultant exhaust emissions. In many urban areas motor vehicle emissions are responsible for emitting major portions of hydrocarbons (HC), carbon monoxide (C(), and nitrogen oxides (NO_x) into the environment (11). Among the factors that affect vehicle emission characteristics are the mix of vehicles (varying by model year) in a particular roadway, the density of vehicles, and the velocity profile of the traffic stream (12). From the individual vehicle velocity profiles, one can ascertain the amount of accelerating, decelerating, and stopping modes experienced by this vehicle.

It has been well established that emission rates vary according to the different rates of acceleration and deceleration of a vehicle's trajectory. According to Noll (13), there is a four fold increase in carbon monoxide emission when the traffic speed changes from 15 to 7.5 miles per hour. See Figure 4. By increasing the average speed of the

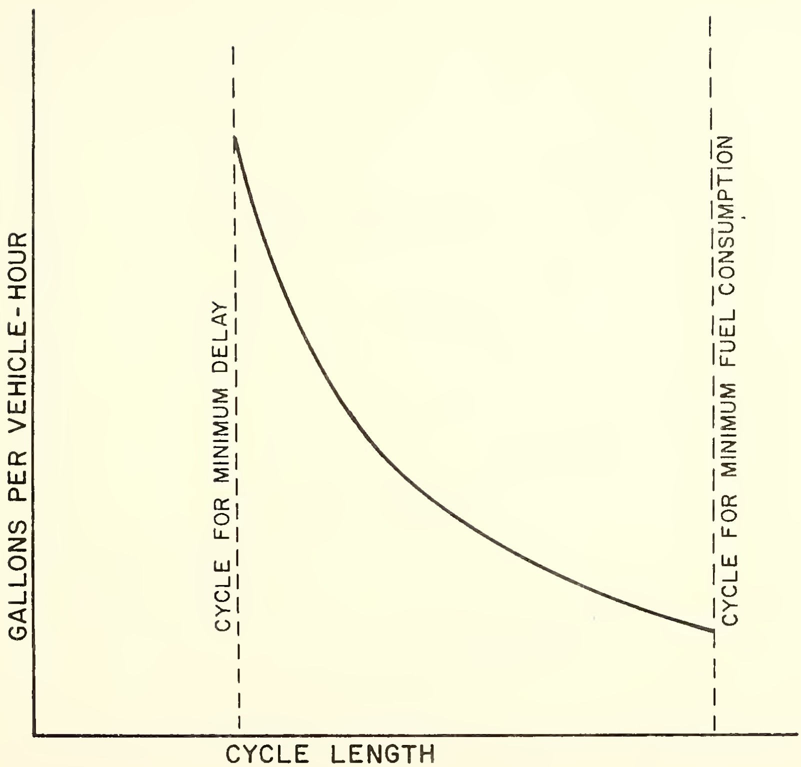

FIGURE 3: FUEL CONSUMPTION AS A FUNCTION OF CYCLE LENGTH
(SOURCE: REFERENCE 10)

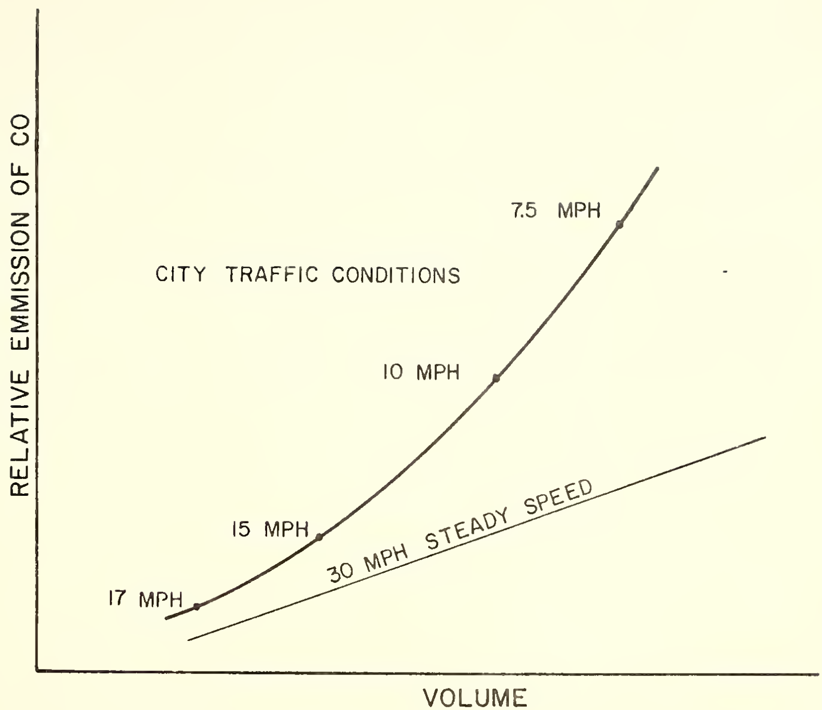

FIGURE 4 EFFECTS OF DISTURBED TRAFFIC FLOW
(SOURCE: REFERENCE 15)

traffic stream through a series of signals, one can generally expect to decrease pollutants emitted by these vehicles.

Earlier research has indicated that significant improvements to traffic flow on a signalized street system can be obtained by simply retiming existing signals. Expensive computerized traffic controllers are often unneeded. Joint regulations (TSM) issued by the Urban Mass Transportation Administration and the FHWA in 1975 require that urban areas more fully utilize their existing equipment and systems before federal assistance is sought for the development of additional facilities (14). These stipulations recognized that financial and non-renewable resources are continually being depleted while demand for travel continuously increases.

The engineer should strive for cost effective controls that interrupt the flow of traffic as little as possible. Traffic flowing smoothly on arterial streets will exhibit a favorable profile of reduced delay, increased fuel efficiency and lower amounts of exhaust emissions.

CHAPTER 2: SIMULATION MODEL DESCRIPTION

In order to conduct efficient analyses, one must employ useful tools. The goals of good traffic engineering can be more effectively realized by advanced decision-making techniques. The effects of specific policies or formulations should be known prior to arriving at a decision. The traffic engineer can 1) implement the policy on a temporary basis and then study its effects; 2) using previous experience and state-of-the-art methods select a specific alternative after careful intuitive analysis, or 3) employ computer simulation capable of analyzing a multitude of alternatives by comparing various measures of effectiveness (MOE). Computer simulation was applied in this research because of its rapid, cost effective characteristics that provided adequate information. Without simulation, estimates of gasoline consumption and automobile emissions would be difficult to obtain. Because of its availability and structure which conforms to the needs of this research effort, the UTCS model was employed as the analysis tool.

UTCS-1 Description

UTCS-1 was designed as an instrument to test control strategies on an urban street network. The model has the capability of examining a full range of traffic control schemes including signal phases, offsets and splits, turn prohibitions, lane additions, as well as unsignalized intersections. Also included in the model's capability is the simulation of pedestrian interference, effects of roadway grade on traffic flow, and the effects of traffic composition i.e., trucks and buses, on traffic flow.

Available from the FHWA, UTCS-1 and its smaller counterpart UTCS-1S have sufficient flexibility to be of ideal use in this study. UTCS-1S, because of its less expensive operation was modified for use herein.

UTCS was developed by several organizations under contract of the FHWA. It has since been validated in many areas of the country and is being increasingly used as a traffic analysis tool.

As modified by Hall in his work at Purdue University (16), the UTCS model used in this study collects and stores individual vehicle speed profiles for one intersection. This is necessary in order to calculate the fuel consumption and vehicle emissions at the intersection of interest. A supplementary computer model developed under the auspices of the Environmental Protection Agency (EPA) computes both individual vehicle fuel consumption and emissions as well as calculates the average fuel consumption and emission statistics for the modeled intersection. Output from the UTCS model is used as input to the EPA model.

Features of UTCS

The current edition of the UTCS model at Purdue University has the capability of representing 60 intersections. Written in FORTRAN, this version needs approximately 145000 central memory units on CDC equipment. It is operational on both CDC and IBM computers. A wide range of geometric configurations can be modeled with provisions for a flexible mix of input and output options (17). The street network of interest is modeled by segmenting homogeneous sections of streets (links) and connecting them by intersections (nodes). Appendix A provides a detailed summary of input instructions and also a sample simulation run of a hypothetical street network.

In addition to the aforementioned, the UTCS model has the following features (18):

1. Detailed treatment of both intersection and link behavior, including queue build-up, queue dissipation, identification of spillback, turn movements, and gap acceptance.
2. Provision for use of default options and distribution values.
3. Output of useful MOE's including average link speed, vehicle delay, network occupancy.
4. Explicit error messages facilitating error detection.

5. Stochastic simulation of individual vehicle by type using a 1 second memorandum notation.
6. Utilization of a simplified car-following model.

Among the set of control options which may be examined by the model are (19):

independent, fixed-time traffic signals

coordinated systems of fixed-time traffic signals

vehicle actuated traffic signals, including semi-actuated, fully actuated and volume density control.

STOP and YIELD sign control

one-way streets

parking prohibitions

lane and turn controls

left and right turn pocket

modifications in roadway geometry, including special channelization at selected intersections.

The UTCS model has a number of distributions embedded in the program. Many of these distributions are entered in a random fashion using a random number generator included in the program. Table 1 summarizes the data included in the program. These data sets are standard traffic engineering data (20) and may be altered at the discretion of the user.

Table 2 lists the major data inputs needed to run the UTCS program. As can be seen upon examination of the necessary data, one can easily ascertain the needed data from traffic engineering records and from a certain degree of familiarity with the street system to be modeled. The input data is specified on specific card types, namely, identification cards, link cards, signal cards, flow rate cards, and control cards. Table 3 contains the standard output statistics for individual links (streets) as well as network-side statistics. These measures of effectiveness are statistically compared with other alternative control options to determine a specific proposal's viability.

Table 1. Summary of Embedded Data

- Distribution of leading and lagging left turn probabilities.
- Distribution of amber phase response.
- Distribution of acceptable gaps for traffic discharging from a Stop sign.
- Distribution of acceptable gaps for left turning vehicles at signalized intersections.
- Value of acceptable lag for lane-switching.
- Intersection turning speeds.
- Maximum number of links in the network.
- Maximum number of nodes in the network.
- Maximum number of entry links in the network.
- Mean effective vehicle lengths, by vehicle type.
- Maximum number of vehicles accommodated on the network at any given instant.
- Distribution of spillback probabilities.
- Distribution of pedestrian conflict delays, by type of flow and type of interaction.
- Distribution of intra-link target speeds.
- Distribution of queue discharge headways.
- Distribution of lost-time for first vehicle in queue.

Source: Reference (21).

Table 2. Summary of Data Input

- Network geometry by link: length in feet, number of moving lanes, grade (%).
- Mean target speed by link in miles per hour, determined by average speed if conditions were ideal. Typically 20-30 mph.
- Mean discharge rate by intersection approach in seconds per vehicle usually taken as 2.1.
- Mean lost time by intersection approach in seconds. An accepted value for this is 3.7.
- Intersection turning movements (%).
- Capacity of left and right turn pockets in vehicles.
- Lane use channelization and turning controls.
- Pedestrian flows on each intersection approach in pedestrians per hour.
- Signal phases, splits, and offsets.
- Identification of stop or yield sign controls.
- Intersection approach volumes in vehicles per hour.
- Traffic composition (% trucks).
- Maximum initialization time for loading network in seconds. Usually about 500 seconds is sufficient.

Source: Reference (22).

Table 3. Standard UTCS Output

Link by Link Statistics

- Link identification.
- Vehicle miles traveled.
- Vehicle trips.
- Total delay per vehicle in seconds.
- Ratio of moving time to total trip time.
- Number of cycle failures
- Space mean speed in miles per hour.
- Average saturation percentage.
- Average number of stops per vehicle.
- Queue delay per vehicle in seconds.
- Average link occupancy.

Network-Wide Statistics

- Total vehicle miles traveled.
- Total travel time in vehicle-minutes.
- Total vehicle trips.
- Average number of stops per vehicle
- Total delay in minutes.
- Space mean speed in miles per hour.
- Average delay per vehicle in minutes.
- Mean network occupancy.
- Queue delay in minutes.

UTCS Structure

The UTCS simulation model is based primarily on two previously developed network simulation models: the "DYNET" model developed by Lieberman and "TRANS" developed by Gerlough and Wagner (23). UTCS is the most complex and detailed of the three.

The UTCS model is divided into three major parts that perform specific functions as controlled by a main program. Figure 5 depicts the relationship between the main program and the two overlay structures. (The UTCS version at Purdue University, unlike the original FHWA version, has only two overlays. The third overlay, Post-Processor Module, was removed without affecting simulation results.)

The Pre-Processor Module checks the consistency of the input data. Helpful diagnostic reports are printed if an input error is encountered. This module also provides the capability of successive simulation runs based on the modification of input data. If an error is detected by the pre-processor, the diagnostic report is printed and the run aborts.

The UTCS-1 Simulator consists of 30 subroutines which are automatically called upon as needed by the specific network under investigation. The simulator requires as input a detailed description of the street system as previously described. The traffic control option being analyzed as well as input volumes are required as input. As output, the simulator lists link-specific and network-wide MOE's.

The movement of traffic through the network is simulated so as to treat each vehicle separately. Each vehicle's movement is governed by its performance characteristics assigned probabilistically as the vehicle enters the network (24). A series of microscopic car-following, queue discharge, and lane switching algorithms further govern each vehicle's motion (26). The model is operated over a sequence of one second time increments. Vehicles are initially emitted onto the network as specified by volume inputs by entry nodes. All vehicles on the network are processed once every second. A random number (0 to 9) is assigned each vehicle upon entry. Using this number, various decile distributions are accessed to determine a particular vehicle's amber phase response, turning speed, and other characteristics.

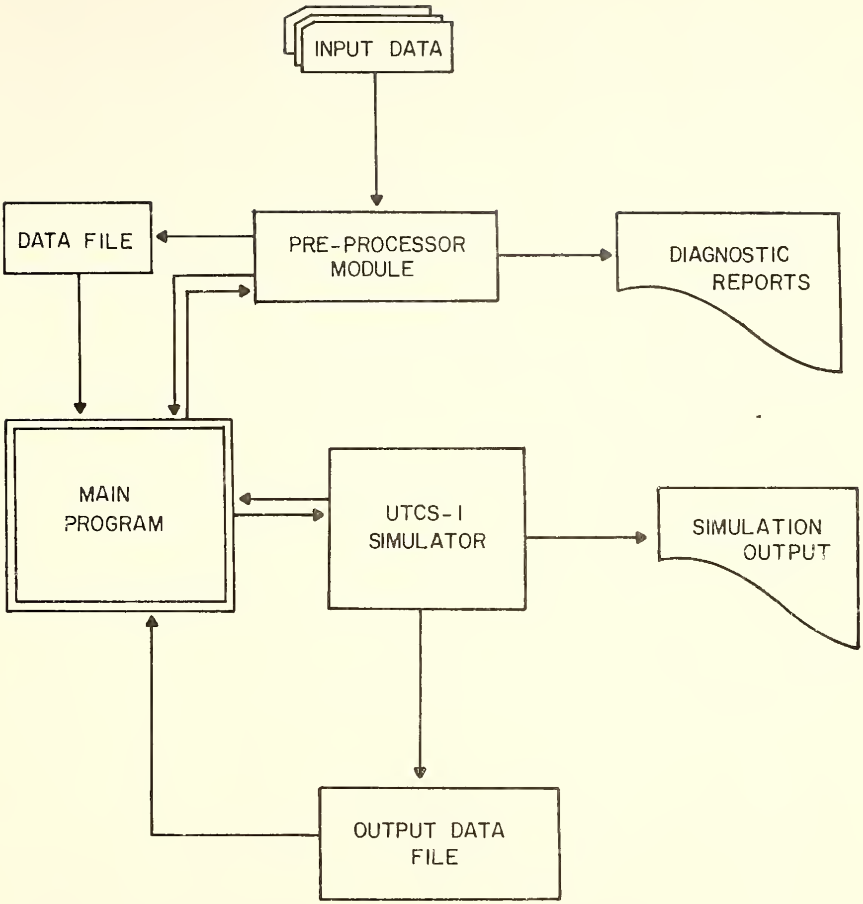

FIGURE 5: OVERLAY STRUCTURE OF UTCS-I
(SOURCE: REFERENCE 24)

A detailed array is maintained for each vehicle including its statistical history as it moves through the simulated network. This array contains: type of vehicle, cumulative trip time, travel distance, cumulative delay, number of stops, and projected turning movement at the next intersection. All information in this array is updated every second. The data contained in this vehicle array is used by the various other subroutines and is also the basis for computation of link and network statistics (27).

Before simulation begins, the network is void of vehicles. The program provides for time to initialize or fill the network with vehicles allowing real-world conditions to be more closely modeled. Simulation will commence once this equilibrium is attained. The program checks for equilibrium by comparing the number of vehicles emitted onto the network with the number of vehicles leaving the system. Close correspondence assures equilibrium. Satisfaction of this standard permits simulation commencement and the accumulation of statistics.

Simulation Logic

Within the subroutine SIMUL, a central control loop performs the simulation for the user-specified time interval. Figure 6 illustrates the traffic simulation loop formed by the various subprograms within the subroutine SIMUL. This loop is performed for each second of the desired simulation interval.

The UTCS program utilizes memorandum notation to represent the flow of traffic by computer. This notation uses an entire word to represent a vehicle and its individual characteristics including time of entry into the system and its gap acceptance features. Each vehicle is considered a separate identifiable entry making it possible to calculate a vehicle's delay and travel time. Distance along the street network is represented in terms of unit blocks each one lane wide with a length equivalent to a portion of the length of an average vehicle. This delineation allows the vehicle to occupy only a limited number of discrete positions in core storage. Movement of the vehicle along the roadway is accomplished by changing the record or file to

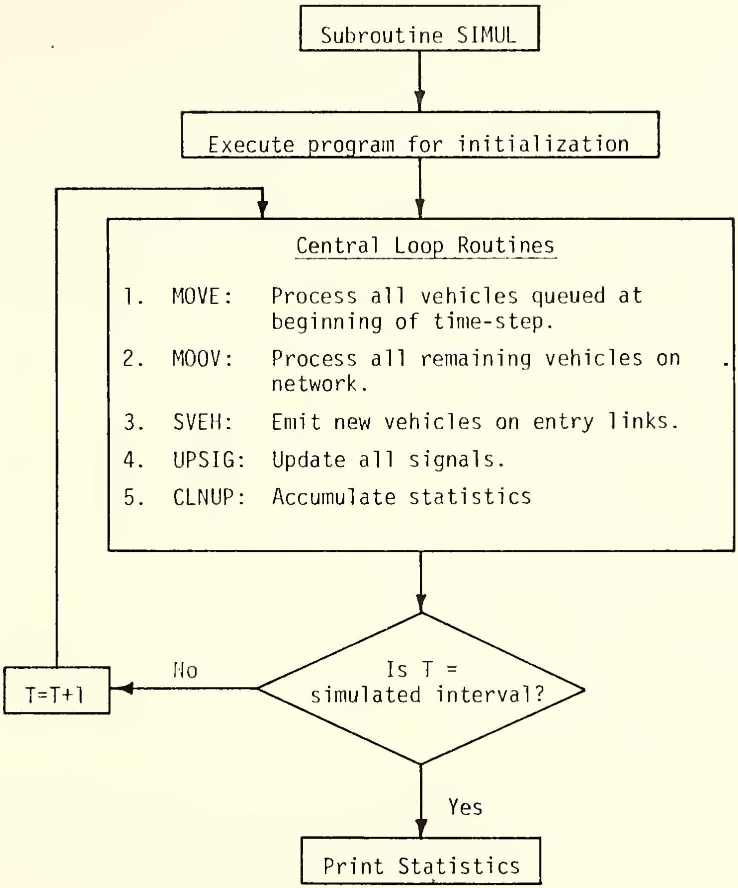

Figure 6. Central Simulation Control Loop

show the vehicle's position one time unit later (1 second). The current speed of the vehicle is multiplied by the time increment and the product is added to the vehicle's present position (28).

Previous Study Using UTCS

The UTCS model was originally calibrated using the downtown area of Washington, D. C. as the test network. Since then several changes have been made to the model to better represent the vastly complex behavior of traffic. Sensitivity tests have indicated that the model "responded to variations in the input conditions in a consistent and intuitively realistic manner "(29). Extensive field data was collected from the Washington, D. C. network using photography to accurately determine headways, travel speed, lane changing criteria and other needed information.

Two other traffic oriented computer models, SIGOP and TRANSYT, develop signal timing schemes given a network configuration (30). UTCS was employed to test each model's efficacy by using the generated traffic control schemes of the two models. The results of this study further illustrate the adaptability of the UTCS simulation model. Actual tests aimed at determining the model's validity are presented in Chapter 3.

Modifications to UTCS-1S

The UTCS-1S model is a smaller version of the full network simulation model, UTCS-1. It was reduced in size for more efficient analysis of one intersection or a mini-network containing only a few intersections (31). The size reduction was accomplished by reducing the core space set aside for various arrays which included the maximum number of vehicles on the network, the maximum number of links and nodes and the maximum number of entry links. For the purpose of this research, the dimension statements were increased to handle up to 85 links, 60 nodes, and 400 vehicles. Specific information concerning alteration of the basic model size parameters is contained in Volume 4 of the UTCS Manual (32). Most statement changes occurred in COMMON statements where array sizes were specified.

Subroutine CLRALL contains data which primes storage with the embedded calibration data. This routine also clears all COMMON storage arrays. For more precise simulation results one may more accurately depict driver idiosyncracies from different geographic areas. Card Types 46 through 54 may be included in the data deck to change this embedded data.

The array TIMVEL, as added to UTCS by Hall, had a capacity of 60 vehicles to accumulate statistics for calculation of energy consumption and vehicle exhaust emission. The array size was increased to 100 because of the increased volume traversing the intersection in this study. It is this array that is used as input to the EPA exhaust emission modal analysis model.

Energy and Emission Model Description

The "Automobile Exhaust Emission Modal Analysis Model" developed under contract for the Environmental Protection Agency (EPA) was used as a supplemental tool in this research effort (33). As modified by Hall (34) this model will output average miles per gallon and emissions in grams per mile of the following pollutants: hydrocarbons (HC), nitrogen oxides (NO_x), carbon monoxide (CO), and carbon dioxide (CO_2). As originally developed and validated, the EPA model only specified exhaust emissions given each vehicle's velocity profile.

By knowing the carbon-based products emitted by each vehicle one can calculate the quantity of fuel consumed using the following expression (35). By relating the amount of carbon in a gallon of fuel to the quantity of carbon compounds emitted, a carbon balance may be expressed. Incidental emissions prior to tailpipe exit are ignored.

$$\text{mpg} = \frac{\text{grams of carbon/gallon of fuel}}{\text{grams of carbon in exhaust/mile}}$$

A further refinement of this relationship was incorporated by Hall into the modal analysis model to estimate individual as well as overall fuel consumption.

Input

Emission rates for various vehicle types are functions of the speed and acceleration profiles of a vehicle through the study area. By segmenting this driving sequence into homogeneous categories of speed and acceleration, these operating categories or "modes" can be analyzed using emission parameters. By knowing empirically derived emission parameters, one can calculate emissions from a vehicle for its various operating modes.

Vehicle emission data are provided for a myriad of light duty vehicles in accordance with model year, engine size, manufacturer, accumulated mileage, state of maintenance, pollution control devices, and geographic location (36). The UTCS program probabilistically assigns characteristics to vehicles according to the percentage of vehicle types in the Indiana vehicle population.

As specified by the user the UTCS model will store individual vehicle speed profiles for two approaches at one intersection. Statistics are accumulated for these vehicles as they enter the intersection, carry out their predetermined turning movements, and exit the observation area. When modeling a number of intersections, one is able to obtain energy and emission statistics for only one intersection at a time. To analyze any network, inherent inefficiencies in the simulation process would surface because of the necessity of modeling the same area a number of times equivalent to twice the number of intersections. Since only two approaches to any one intersection can be modeled for any one simulation run, two simulation runs are needed to determine one intersection's energy and emission characteristics. For this reason, a relationship between traffic engineering measures of effectiveness such as average speed and the energy and emission statistics for one intersection was developed and applied to the entire network. These regression equations and their applicability to this study are derived in Chapter 4.

By deriving these regression equations using both the EPA and UTCS models, fuel consumption and carbon monoxide emissions could be calculated exogenously, sacrificing little accuracy. This method proved to be much more economical in that it minimized the use of expensive computing time. The practicing traffic engineer can also easily derive fuel and emission measures when given the independent variables hereinafter specified.

CHAPTER 3: ANALYSIS MODEL VALIDATION

Both the UTCS-1 and the EPA Exhaust Emission Models have been carefully developed, calibrated and verified with field data collected under varying conditions. Because of the national distribution of automobiles, coefficients developed to analyze their exhaust based on velocity profiles will vary little among low altitude cities. Individual driver characteristics i.e., gap acceptance and distribution of speed given a target velocity will vary depending upon the specific locale being modeled. For this reason, it was desired to further substantiate the output of the UTCS model for local conditions and driving behavior. This chapter reviews the previous efforts involving the UTCS-1 simulation model then summarizes the validation procedure in a small city in Indiana. Verification of the EPA model's output accuracy is presented based on past efforts. Both models have proven to be realistic in their estimates of traffic flow measures of effectiveness.

Past Validation of UTCS

The UTCS simulation model was developed in two phases. Phase I included the calibration and validation of the basic simulation program against a myriad of field data collected from a test network in downtown Washington, D. C. Phase II provided for a series of revisions to more accurately simulate the complex flow of traffic. Demonstration and sensitivity tests for both intuitively reasonable and mathematically rigorous behavior of the traffic stream were conducted. Default calibration inputs were added to the model in Phase II so that one could, without specific locale validation, employ the model to test alternative traffic control techniques using only a relative scale. If necessary the user may alter these calibrated input data to more accurately replicate a street network.

Logic was revised or incorporated to correctly model intersection queue discharge, vehicle and driver behavior at four-way stop signs, acceleration characteristics of automobiles and trucks on grades, and behavior at signals permitting a right turn on the red indication. Intersection turning movements are potentially hindered by pedestrian interference. This delay caused to vehicles is determined stochastically from a distribution which is dependent upon the amount of pedestrian traffic at a given location.

Acceleration rates were applied separately to cars and trucks under all conditions. Although a grade would adversely affect a truck's rate of acceleration, the model developers thought this not too significant because of the low speeds typical for urban streets (37). The model applies the acceleration rate of 8 ft/sec^2 until the car reaches a speed of 16 ft/sec (11 mph); then the acceleration rate is reduced to 3 ft/sec^2 until the assigned speed is attained. Under all conditions the acceleration rates for trucks was specified at 3 ft/sec^2 until the truck reaches the speed of 16 ft/sec; thereafter the acceleration rate is 2 ft/sec^2 up to the desired speed. Several independent studies confirm the validity of many of the calibration standards used in building the UTCS simulation model.

Other studies, after individual validation, used UTCS as an analysis tool in examining several traffic control techniques. Also, the model has been used without specific validation for analysis of alternatives on a relative scale. In a street network in Washington, D. C. where the model was calibrated, a validation exercise was conducted. In the morning peak traffic conditions, the simulation model reproduced the actual traffic performance on the network quite accurately (38). Seven measures of effectiveness (total link output, average network occupancy, vehicle minutes, vehicle miles, total delay, average travel time, and average speed) for simulation and field observation were compared. Discrepancies between field and model performance averaged below 2.4 percent (39). The accuracy of the simulation model decreases somewhat during periods of lighter undisciplined traffic flow. The model tends to impede traffic less than actual conditions and thus underestimates the average travel time.

Further utilization of UTCS was gained in modeling a simple linear arterial to test signal timing schemes. Again the model reproduced field data and demonstrated its usefulness as a tool for traffic engineers in evaluating potential improvement methodologies (40). Another study worthy of note is one made in San Jose, California (41). This study was primarily concerned with illustrating the model's capability by simulating traffic conditions different from the original Washington, D. C. calibration network. Its usefulness as an analysis medium was also demonstrated in that two computer models SIGOP and TRANSYT, which generate signal timing plans, were compared for their degree of effectiveness (42).

A study conducted by the Utah Department of Transportation used the UTCS model and validated it on the basis of vehicle counts. It was concluded from this investigation that the idiosyncracies of Utah drivers were similar enough to the driver characteristics in the Washington, D. C. validation network to consider the model's output representative of the street network in Utah. A paired t-test was used to compare the differences between observed vehicles counts and the simulated vehicle counts. This test showed no significant difference at the 10% significance level (43). Further evidence is thus provided that the model is indeed capable of accurately replicating actual traffic conditions.

These studies substantiate the credibility of the model and suggest that its use for simulating local traffic problems is conducive to optimizing traffic flow. Even if the real values of all the measures of effectiveness computed by the model cannot be verified, one can rely on the relative values of these measures for evaluating traffic control alternatives. An exercise is conducted below to compare the simulated values of local traffic conditions in West Lafayette, Indiana to its observed conditions.

Local Comparison of UTCS

In order to get some idea as to the preciseness with which the UTCS model replicated actual traffic conditions, traffic data was collected. Because of the problem of measuring many of the measures of effectiveness listed in the UTCS output only a few were chosen for the comparisons, namely, vehicle trips, the number of stops per vehicle, stopped delay, and stopped delay per vehicle.

Data Collection

Four intersection approaches were randomly chosen and data was taken at these locations during the morning peak period. Using a method described in Reference (44), the number of vehicles stopped on an approach was recorded every four seconds. At the same time, the volume traveling in the direction observed was recorded. Counts were taken during five 5-minute periods. Because of the high peaking characteristics only one approach could be observed every day, since it was intended to take each 5 minute sample from the same population.

Since most every queue cleared during the green interval, it was also possible to determine the number of cars that had stopped on the approach. For the queues that did not completely clear and for those vehicles that moved up to the intersection then stopped again to wait for an acceptable left turn gap, a special note was made so that the number of stops per vehicle could be accurately estimated.

After the data was collected and collated, the appropriate parameters were calculated and compared to the simulated values. The equation for the total stopped delay on a link in one direction is

$$SD = NCAR \times SI/60$$

where:

SD = stopped delay in vehicle minutes.

NCAR= total number of vehicles stopped during the sampling interval.

SI = sampling interval in seconds (SI = 4)

Average stopped delay per vehicle in seconds is expressed as

$$ASD = 60\ (SD)/V$$

where:

V = number of vehicles counted during sampling period (5 minutes).

The number of stops per vehicle can be determined by

$$SPV = NCAR/V$$

Five replications were run on the computer using a different random number generator seed for each replicate. Tables 4, 5 and 6 show the observed $\pm$ standard deviation as well as the simulated values for stops per vehicle, stopped delay for the approach, and stopped delay per vehicle, respectively.

To check the model's accuracy for discharging vehicles onto the network and distributing them among the links according to preassigned turning movements, volume counts were taken and statistically compared to the simulated counts. See Table 7.

Statistical Analysis

The student's t-test was used to compare the observed and simulated delay measurements for determination of the model's accuracy. A goodness of fit test employing the chi-square distribution was used to compare actual and simulated volume counts for the four unidirectional links.

If the simulated counts of traffic are close to the corresponding expected (actual) volume counts, the χ^2 value will be small indicating a good fit. Given the mean of three 25 minute intervals of simulation and a 25 minute actual vehicle count, the χ^2 test was conducted. Table 8 shows this statistical comparison that indicates a good agreement between simulated and actual volume counts.

In order to employ the student's t-test for comparing the three other measures, the requirement of homogeneity of variance had to be met. Testing the equality of variances at the significance level of 0.20 showed that all but one pair of variances for the corresponding

Table 4. Stops Per Vehicle for Observed and Simulated Data

	Observed $\pm$ (s.d.)*	Simulated ($\pm$ s.d)*
Approach A	1.06 $\pm$ 0.16	1.08 $\pm$ 0.15
Approach B	0.88 $\pm$ 0.34	0.81 $\pm$ 0.17
Approach C	0.73 $\pm$ 0.27	0.84 $\pm$ 0.15
Approach D	0.54 $\pm$ 0.08	0.74 $\pm$ 0.15

Note: The approaches observed were randomly selected from among all of the approaches on the Northwestern Avenue arterial.

* The s.d.'s represent the calculated s.d. for each set of data. A pooled estimate of the variance should be used if one is going to place a confidence interval on the means.

Table 5. Stopped Delay by Approach in Minutes

	Observed $\pm$ s.d.	Simulated $\pm$ s.d
Approach A	40.0 ± 14.2	67.6 ± 15.0
Approach B	22.82 ± 13.9	19.3 ± 8.7
Approach C	8.3 ± 2.8	8.7 ± 1.8
Approach D	23.5 ± 5.9	20.0 ± 2.4

Table 6. Stopped Delay per Vehicle by Approach in Seconds

	Observed $\pm$ s.d.	Simulated $\pm$ s.d.
Approach A	24.8 ± 8.9	41.1 ± 9.7
Approach B	32.6 ± 20.0	26.3 ± 10.3
Approach C	21.6 ± 7.4	23.9 ± 6.6
Approach D	16.6 ± 4.2	15.1 ± 2.6

Table 7. Vehicle Counts by Approach

	Actual (expected)*	Simulated
Approach A	480	484
Approach B	208	213
Approach C	115	109
Approach D	407	409

Note: Vehicle counts are for a 25 minute period during the AM peak hour on the Northwestern Avenue arterial.

*These volume counts are the actual counts taken on the approaches. For purposes of the χ^2 test these are referred to as the "expected" values.

Table 8. Statistical Comparison, Actual (Expected) and Simulated Volume Counts

$H_0: V_{exp} = V_{sim}$ $H_1: V_{exp} \neq V_{sim}$ $\alpha = 0.25$

$$\chi^2 = \sum_{i=1}^{n} \frac{(V_{isim} - V_{iexp})^2}{V_{i_{exp}}}$$

$$\chi^2 = \frac{(484-480)^2}{480} + \frac{(213-208)^2}{208} + \frac{(109-115)^2}{115} + \frac{(409-407)^2}{407} = 0.476$$

$$\chi^2_{3,\alpha} = 4.11$$

observed-simulated data were different. This pair of variances was not statistically different at the α level of 0.10. Given that slight departures from the homogeneity of variance requirement will not seriously affect the t-tests, the analysis continued. The accuracy of the results was enhanced by keeping the sizes of samples equal.

Each measured parameter was tested against its corresponding simulated value at a level of significance (α) of 0.40. A sample calculation of this test is shown in Table 9 and the results of these tests for the randomly selected approaches and the three measures of effectiveness are tabulated in Table 10. A point estimate of the unknown common variance σ^2 was obtained by using a pooled variance, $S_p^{\ 2}$.

Since it is desired to fail to reject the null hypothesis, one must consider the probability of committing a Type II error - accepting the null hypothesis, H_0, when it is false. This probability is denoted by β and is shown in Table 10 along with the alpha values at which each test was conducted. The value of β is impossible to compute unless a specific alternative hypothesis is specified and as such it was desired to detect differences in the true population of each of the measured parameters by the following values:

stops per vehicle	0.20 stops per vehicle;
stopped delay	10.0 minutes;
stopped delay per vehicle	10.0 seconds.

Given these values, one could then calculate the value of β for each test conducted at an alpha level of 0.40. Since an increase in the level of significance generally results in a decrease in the probability of committing a Type II error, the tests were conducted at a relatively high α.

The observed data cannot be taken as the exact actual conditions since, no doubt, some error was introduced in counting. It must be recognized also that since the tests were conducted at relatively high alpha levels (α = 0.4), the probability of rejecting the null hypothesis ($\overline{X}_{sim} = \overline{X}_{obs}$) when it is, in fact, true,is rather high. This condition, however, does not detract from the usefulness of the analysis.

Table 9. Sample Test Calculation - Observed and Simulation Data, Stops Per Vehicle on Approach A

$H_0: \bar{X}_{sim} = \bar{X}_{obs}$ $\qquad$ $H_1: \bar{X}_{sim} \neq \bar{X}_{obs}$ $\qquad$ $\alpha = 0.4$

$$t = \frac{X_{sim} - X_{obs}}{S_p\sqrt{\frac{1}{n_{sim}} + \frac{1}{n_{obs}}}}$$

where $$S_p = \sqrt{\frac{(n_{sim} - 1)\, S^2_{sim} + (n_{obs} - 1)\, S^2_{obs}}{n_{sim} + n_{obs} - 2}}$$

$$t = \frac{1.08 - 1.06}{0.155\sqrt{\frac{1}{5} + \frac{1}{5}}} = 0.204$$

$$t_{8,\alpha/2} = 0.889$$

Table 10. Summary of t-Tests for Observed and Simulated Data

$H_0: \overline{X}_{sim} = \overline{X}_{obs}$ $\alpha = 0.40$

Approach	Stops Per Vehicle	Stopped Delay	Stopped Delay Per Vehicle
A	$t = 0.204$	$t = 3.0$*	$t = 2.76$*
	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$
	$\beta = 0.12$	$\beta = 0.39$	$\beta = 0.20$
B	$t = 0.40$	$t = 0.48$	$t = 0.63$
	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$
	$\beta = 0.35$	$\beta = 0.29$	$\beta = 0.42$
C	$t = 0.80$	$t = 0.26$	$t = 0.52$
	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$
	$\beta = 0.27$	$\beta < 0.001$	$\beta = 0.10$
D	$t = 2.63$*	$t = 1.22$*	$t = 0.675$
	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$	$t_{8,\alpha/2} = 0.889$
	$\beta = 0.05$	$\beta = 0.01$	$\beta = 0.003$

*Significant difference in $\overline{X}_{sim}$ and $\overline{X}_{obs}$.

Referring to Table 10, one can see the degree to which the simulation model reproduced actual traffic flow parameters. Each parameter must be considered separately as the three measures are not independent of each other. Because of the small sample size, one cannot, on the basis of this validation test only, conclude that the entire model is adequate. However, the model did indicate an acceptable fit between the observed and simulated data for the approach links observed. Based on previous validations, the output of this analysis further lends credence to the soundness of the model.

Verification of the EPA Modal Analysis Model

In 1971, the Environmental Protection Agency developed the Surveillance Driving Sequence (SDS) to measure vehicle emissions over a variety of constant speed and variable acceleration conditions. These acceleration and deceleration modes as represented in the SDS are composed of the many combinations of five speeds: 0 mph, 15 mph, 30 mph, 45 mph, and 60 mph. Modal analysis is achieved by utilizing input data of the emissions measured from 37 distinct modes of the SDS (45). By expanding these discrete modes into a continuous regression response function, it is possible to integrate the emission rate function over to the velocity profile of the vehicle in question (46). Given the emission rate function for a vehicle and a pollutant, a vehicle's response can be ascertained. Each vehicle is characterized by 48 parameters or coefficients, 12 of which are for the specification of each emission rate function used in estimating the quantity in grams/mile of NO_x, CO, CO_2, and HC emitted by a vehicle. Emissions measured over the Surveillance Driving Sequence are those from a warmed-up vehicle (47). The total amount of a particular pollutant emitted by a vehicle in a given driving sequence is often called the "bag value".

The input coefficients as updated by Hall (48) allowed pollutants to be estimated for pre-controlled autos (1967 and earlier) through 1975 model light duty vehicles. Also, deterioration factors were applied to account for wear on the autos originally tested. Values for the coefficients have been determined by the least squares method of fitting rate functions to empirically derived test data (49). This technique forms the basis for relating experimental observations to the specification of an emission rate function.

By comparing model output with measured quantities, one can evaluate the performance of the model. In development and refinement of the EPA model, many validation attempts were undertaken. However, there are many variables affecting the levels of air pollution, and some of these variables are difficult to control. Surrounding environment, climate, and wind conditions largely affect the concentration of emitted pollutants. Other discrepancies can be attributed to the variations within model year and to a vehicle's condition. Consequently, precise validation of the model's results is not always possible. To date, however, this EPA model represents the best disaggregate emissions analysis tool available (50). This analysis aid has been shown to accurately simulate relative differences in emissions from vehicles exhibiting differing velocity trajectories. Thus one can look at energy savings and pollution reductions to determine the relative desirability of alternatives under consideration. A vehicle's velocity profile in one second steps, model year, energy consumption, and emissions are shown in Figure 7 as an example of the EPA model output.

While the EPA model output was not directly used in developing efficient signal timings, this research employed the model to develop relationships between a network-wide average fuel consumption and the resultant carbon monoxide emitted to the atmosphere. Of particular interest in this research was the amount of carbon monoxide discharged by the traffic stream under various flow conditions effected by signal timings. The correlation of fuel consumption and grams per mile of carbon monoxide emitted by vehicles disaggregated by model year is the subject of the following chapter.

Auto No. 133 Model Year = 1970

Speed-Time Profile =

22.500	22.500	22.500	22.500	21.818	21.136	20.455	19.773	19.091	16.364	16.364
18.409	20.455	22.500	22.500	22.500	22.500	22.500	22.500	22.500	22.500	22.500

Emissions in Grams =

HC = .655 CO = 7.157 NO_x = .440 CO_2 = 50.372

Miles per Gallon = 17.180

Gallons of Gasoline Used = .00718

Figure 7. Sample Individual Vehicle Output of EPA Model

CHAPTER 4: MEASUREMENT OF FUEL CONSUMPTION AND CARBON MONOXIDE EMISSIONS

As previously mentioned the modified EPA model is used jointly with the UTCS model to accumulate statistics for calculation of vehicle fuel efficiency and emissions. In this computer package only one intersection may be specified at a time for determination of its energy and pollution characteristics based on the individual vehicle velocity profiles traversing the user-specified intersection. These profiles, which are output from UTCS on a file named TAPE18, must be stored by the user to be utilized as input to the EPA model for further analysis. If the objective of this study was to investigate single isolated intersections the UTCS-EPA models as modified would be ideal. In order to deal with a system of intersections, the subsequent development was undertaken.

The EPA model was used to develop a relationship between fuel consumption and carbon monoxide emissions. Indeed, the energy and pollution characteristics of a number of intersections analyzed separately would not be the same as the characteristics exhibited by a system of intersections along an arterial. The timing of an upstream signal with respect to a downstream signal would most definitely affect the traffic stream. Spillback of a queue caused by a downstream red signal would hinder the flow at the upstream intersection. The network as a whole must be considered in developing its measures of effectiveness. Predictive equations given a network's measured parameters will provide sufficient information on the network's fuel consumption and carbon monoxide emissions traits. In the following paragraphs are discussed the procedures undertaken to develop the predictive equations.

Prediction of Fuel Consumption

Under FHWA contract, the Honeywell Traffic Management Center (51) correlated network-wide fuel consumption with traffic stream parameters such as average speed, signal cycle length, the number of vehicle trips, and the delay per vehicle mile. This correlation was in the form of a linear regression equation. The Honeywell report used a different version of UTCS which included a provision for storing each vehicle trajectory for an entire network and simulation period on a peripheral storage device and subsequently retrieving this data to determine the average fuel consumption efficiency in vehicle-miles per gallon for the network. The UTCS-EPA models used jointly as described above would be most useful in the study of isolated intersections. Honeywell's treatment was conducted on a much larger scale as an entire network's vehicle trajectories were analyzed.

The Honeywell Traffic Center, through numerous simulation runs of urban arterial streets and street networks, developed correlations between fuel consumption and other network measures of effectiveness. Streets in both California and Washington, D. C. were modelled and actual data was taken on both systems. The study's major conclusion was a regression equation relating fuel consumption to average speed. Using aggregated data from six experiments conducted in the study, it was determined that among the many traditional traffic engineering measures of effectiveness fuel consumption was most highly correlated with average network speed and can accurately be predicted by the equation (52)

$$FC = 3.61 + 0.412S \tag{1}$$

where:

FC = fuel consumption in vehicle miles per gallon.

S = average network speed (space mean speed) in miles per hour.

When the values for fuel consumption predicted by this equation were compared with actual fuel consumption values an average mean square error for the predictor was observed to be 0.559. The R^2 value, which indicates the proportional reduction in the variability of fuel consumption attained by use of average network speed, was found to be equal to 0.92. Speeds taken into consideration for the development of this equation range from about 3 to 23 miles per hour. Prediction of fuel consumption outside this range of average speeds would be questionable.

Another multiple linear regression equation was developed by Honeywell for prediction of fuel consumption which included 10 variables. Even though this more complex equation provides a better estimate the gain in accuracy must be weighed against the increased complexity as well as the need of determining many more measures of effectiveness in the field. The increase in accuracy, assuming precise data collection, is only 6.9% which is very likely less than the expected experimental error. As such, the simple linear regression equation containing the independent variable, average network speed, was used in this investigation.

Prediction of Carbon Monoxide Emissions

In order to ascertain the relative efficacy of a specific traffic control scheme, it was also desired to include some measure of the levels of carbon monoxide emitted by the traffic stream. A vehicle's discharge of carbon monoxide into the atmosphere is a function of its velocity profile on a street network. An interrupted trip with frequent stops and starts so prevalent in today's urban areas will generate much more carbon monoxide than a smooth trip that exhibits a uniform speed profile. Also, carbon monoxide emission is inversely related to fuel consumption as fuel consumption efficiency (mpg) increases, carbon monoxide (as well as hydrocarbons, nitrogen oxides, and carbon dioxide) decreases. This relationship was used to determine a simple linear regression equation with miles per gallon of fuel as the independent variable and carbon monoxide (CO) as the dependent variable.

The general procedure used in developing this relationship is as follows. More specific explanations and statistical tests will be subsequently presented. The EPA model, when given a vehicle's velocity profile and model year as input will calculate the vehicles fuel consumption, fuel consumption rate, and emitted amounts of HC, CO, NO_x, and CO_2. These statistics are for one intersection only. Table 11 is a typical listing of the EPA model output for one simulation run.

A wide range of velocity profiles were generated by specifying various volumes and signal timings for a single intersection. These velocity profiles in turn were input to the EPA model for calculation of vehicle fuel consumption per mile and carbon monoxide emissions in grams per mile. Because of the decreasing trend in carbon monoxide emissions as the model year increases (better pollution abatement equipment), separate regression equations were developed for each model year.

These velocity proviles could be assumed to be somewhat similar to segments of a vehicle's trip through a system of signals. By using the EPA output, regression equations from the fuel consumption statistics and the carbon monoxide emission statistics were developed. Thus, given a particular vehicle group's fuel rate, the carbon monoxide emitted could be predicted.

Once regression equations were developed to predict CO emission on the basis of the information about fuel consumption at a specific intersection, these equations were employed to predict CO emission given the gasoline consumption rate for the entire network as determined by the Honeywell regression equation. The motor vehicle mix by model year was applied to the separate CO equations to obtain a weighted approximation of CO emissions. In order to determine the relative effectiveness of a traffic control strategy an arbitrarily chosen 100 car standard and its resultant emissions was used. The model year composition according to Indiana motor vehicle registrations is given in Table 12.

In developing the regression equations relating miles per gallon with CO in grams per mile for each model year, pre-1967 through 1975, 20 data points were used. These data points were generated by simulating the various intersections in the test network using typical volumes, signal timings, turning movements, and geometric configurations.

Table 11. Emission and Fuel Consumption by Model Year for a Modelled Intersection

Model Year	No. of Vehicles	Emission Data (gms/mile) HC	CO	NO_x	CO_2	Distance Total (Mi)	Average MPG	Gallons Gasoline
1967	42	10.8	134.9	5.6	556.3	5.012	11.060	.45316
1968	23	9.9	136.2	8.1	765.5	2.711	8.781	.30875
1969	19	8.4	109.9	9.3	687.1	2.307	10.014	.23038
1970	21	7.3	87.8	8.1	673.1	2.548	10.638	.23948
1971	15	6.3	92.3	5.8	627.6	1.808	11.199	.16147
1972	35	5.8	89.6	4.4	638.9	4.248	11.119	.38200
1973	33	5.4	70.3	3.3	689.9	3.990	10.857	.36751
1974	38	5.7	82.0	3.8	822.9	4.552	9.152	.49737

Table 12. Indiana Model Year Distribution

Model Year	Cumulative Distribution*	Percentage
75	1.000	9.5
74	0.905	13.5
73	0.770	15.4
72	0.616	12.4
71	0.492	8.9
70	0.403	9.3
69	0.310	7.8
68	0.232	6.0
67		
66		
65	0.172	17.2
64		
≤ 63		

*Weighted by average annual miles driven.

Source: Reference (53).

The individual regression equations are given below.

$$CO(\leq 67) = 214.83 - 6.74\ MPG(67) \qquad (2)$$

$$CO(68) = 198.23 - 7.43\ MPG(68) \qquad (3)$$

$$CO(69) = 192.29 - 8.56\ MPG(69) \qquad (4)$$

$$CO(70) = 169.23 - 7.47\ MPG(70) \qquad (5)$$

$$CO(71) = 211.14 - 10.83\ MPG(71) \qquad (6)$$

$$CO(72) = 203.85 - 10.20\ MPG(72) \qquad (7)$$

$$CO(73) = 177.32 - 9.74\ MPG(73) \qquad (8)$$

$$CO(74) = 169.60 - 9.61\ MPG(74) \qquad (9)$$

$$CO(75) = 131.86 - 9.21\ MPG(75) \qquad (10)$$

where: CO(XX) is the carbon monoxide in grams per vehicle mile for model year XX.

MPG(XX) is the fuel consumption rate in miles per gallon for model year XX.

Table 13 indicates the individual regression equations and their respective R^2 values and mean square error values.

When determining a network's measures of effectiveness with respect to fuel consumption and carbon monoxide emissions, the investigator must first determine the network-wide fuel consumption rate from equation 1. In order to arrive at the 100 car standard for carbon monoxide emissions based on the traffic stream vehicle mix by model year one needs estimates of network fuel consumption rates by individual model year. The network fuel consumption rates for all model years (pre-67-75) could be shown to be from the same population, carbon monoxide by model year could then be determined using only one network fuel consumption rate rather than a rate for each year.

A statistical comparison was employed to determine whether or not network fuel consumption rates varied from year to year for the type of urban driving considered in this report. Using Scheffe's method of multiple comparisons (the most conservative test of Tukey, Student-Newman Keuls, and Scheffe) fuel consumptions rates for each model year were randomly selected and tested. At a level of significance of $\alpha = 0.05$, it was found that the mean network fuel consumption rate did not differ with model year. By substituting network-wide fuel

Table 13. Regression Equations for CO by Model Year

$CO(67) = 214.83-6.74\ MPG(67)$
$R^2 = 0.91$
$MSE = 47.04$

$CO(68) = 198.23-7.43\ MPG(68)$
$R^2 = 0.90$
$MSE = 30.64$

$CO(69) = 192.29-8.56\ MPG(69)$
$R^2 = 0.93$
$MSE = 22.57$

$CO(70) = 169.23-7.47\ MPG(70)$
$R^2 = 0.94$
$MSE = 16.34$

$CO(71) = 211.14-10.83\ MPG(71)$
$R^2 = 0.97$
$MSE = 5.22$

$CO(72) = 203.85-10.20\ MPG(72)$
$R^2 = 0.97$
$MSE = 6.01$

$CO(73) = 177.32-9.74\ MPG(73)$
$R^2 = 0.96$
$MSE = 8.07$

$CO(74) = 169.60-9.61\ MPG(74)$
$R^2 = 0.96$
$MSE = 10.66$

$CO(75) = 131.86-9.21\ MPG(75)$
$R^2 = 0.98$
$MSE = 5.24$

consumption obtained from equation (1), one can estimate the carbon monoxide emissions by model year, then combine each year's emissions weighted by the percentage of vehicles of that year in a typical traffic stream in Indiana. This process is detailed as follows:

1. Determine average network speed(s) in miles per hour.
2. Using FC = 3.61+0.412S, determine the network wide fuel consumption rate (FC).
3. Using equations (2) through (10) and FC determined in the above step, calculate CO (67) through CO (75).
4. For a 100 car standard sum the following for determination of CO emitted per mile:

$$17.2[CO(67)] + 6.0[CO(68)] + 7.8[CO(69)] + 9.3[CO(70)]$$

$$+ 8.9[CO(71)] + 12.4[CO(72)] + 15.4[CO(73)] + 13.5[CO(74)]$$

$$+ 9.5[CO(75)]$$

Using CO, FC, and S plus additional measures of effectiveness available from the UTCS output, one can then determine a system's relative attractiveness. These measures of effectiveness were used collectively to ascertain a specific alternative's performance. Development of these alternatives are presented in the following chapter, while evaluation of these alternatives as to their benefits derived by auto drivers and the surrounding community is detailed in Chapter 7.

CHAPTER 5: SIGNAL TIMING METHODOLOGY

This phase of the project dealt with the task of developing alternative signal timings schemes on the basis of a commonly used set of procedures. While these methods often lead to fairly good timings, one must exercise intuitive judgement based on traffic engineering principles to refine and further improve the signal timing scheme. For the purpose of this study mathematical techniques, both computer programmed and manually calculated, were employed. The study area used for the development of alternative signal timing schemes was West Lafayette, Indiana. Different schemes were prepared based on volume counts for three periods of the day. The UTCS simulation model was then utilized to analyze and compare the operation of these streets under the different control schemes.

It was specifically intended to develop strategies that could be implemented by local traffic engineers without any need for acquisition or installation of additional traffic control hardware. Therefore, the same number of signal phases was considered in the alternative proposals as it exists at present. Although the existing system in most cases consists of fixed-time, interconnected signals with three-dial controllers, the methods developed are not necessarily restricted to these; even computer controlled signals use strategies developed from these methods. Several predetermined signal timing plans are stored in a computerized controller's memory and different strategies are called into operation as volumes and other variables fluctuate (54). Concepts discussed herein should be considered for a wide range of conditions in traffic control systems. Actual results, however, will no doubt vary from system to system, but the overall implications should be of interest for achieving a more efficient traffic movement.

Literature Review

Several methods have been developed to calculate individual intersection cycle lengths and splits. Given individual cycle lengths, one can then determine the offsets from intersection to intersection along an urban arterial with a variety of methods. Strategies found in the literature range in complexity from detailed linear programs to simple two dimensional diagrams used in determining signal offsets along an arterial. Among the methods that have been thoroughly tested and used in this report are:

- Webster Optimization of Cycles and Splits
- Delay/Difference of Offset Method
- Maximal Bandwidth (Time-Space Diagram) Method
  - Half-cycle synchronization
  - Preferential direction

These methods do not yield complete signal plans in themselves and are therefore used in combination with each other. In addition, while these strategies will lead to a good cycle plan, one can refine the plan by examining the analysis data and adjusting the plan accordingly.

Use of computer simulation to analyze the different plans would provide opportunity to try innovative timing schemes; traffic congestion and numerous complaints from motorists will not be the penalty for an idea that failed to perform as expected. In the following paragraphs a brief discussion of each of the procedures considered is presented.

Webster Optimization of Cycles and Splits

F. V. Webster, in a paper published in 1958 (55), set forth a procedure using both theory and computer simulation to derive expressions for cycle length and phase lengths that minimize total delay at an intersection. This method is of importance since it is desired to consider the total delay associated with a linear street system including its minor intersecting streets rather than merely considering the delay on the major street.

For a two phase intersection:

$$C_o = \frac{1.5L + 5}{1-\gamma_A-\gamma_B}$$

where:

C_o = optimum cycle length in seconds

L = total lost time per cycle in seconds (i.e., the sum of lost time of phases at approaches, due to starting delays and reduced flow during the amber phases);

γ_A, γ_B = the maximum ratios of single lane flow to saturation flow for approaches A and B.

Given this cycle length, the green time should be established such that:

$$\frac{GE_A}{GE_B} = \frac{\gamma_A}{\gamma_B}$$

where:

GE_A, GE_B = effective green times of phases A and B in seconds. (Effective green time equals green plus yellow, minus lost time for a given phase.)

In establishing a signal plan for an arterial, the cycle lengths are ordinarily set equal to each other based on the maximum cycle length at the critical intersection. Kreer (56) reports a technique whereby traffic flow may be improved by mixed cycle lengths depending upon individual intersection requirements. If wide discrepancies are evident in the optimal cycle lengths individually, opting for the mixed-cycle mode may be beneficial.

Delay/Difference-of-Offset Method

This technique will result in an offset plan for signals along an arterial using delay as the objective function. The data needed for computation of the offsets using this method include the traffic flows, turning movements, a common cycle length, and green time for each approach. It is assumed that the delay to traffic along a link depends solely on the difference between the offsets of the signals at adjacent intersections (57). Thus one can calculate the delay associated with each offset setting possible (i.e., 0 through n-1 where n is the cycle length in seconds). A computer program to compute the delay value for each possible offset has been written. The actual listing of the FORTRAN program as specified in NCHRP 73 (58) is shown in Appendix B.

The principal assumption for the analysis of a link with signals at its head and tail is that no significant dispersion of traffic platoons occur (59). Since most of the links in the test network are relatively short (less than 1000 feet) this assumption is valid.

Output from the delay/difference of offset program gives the computed values of QSUM - the total delay-in-queue during one signal cycle in vehicle-seconds per signal cycle, DPV - the average delay per vehicle in vehicle-seconds per vehicle, and QAVE - the average queue length in vehicles. These figures are calculated for every value of offset difference in one second time steps. The offset difference between the signals at the head and tail of each link is referred to in the program as PHI. A sample run was made and the output generated is shown in Appendix B, as well as the input data.

The delay/difference of offset technique accounts for dissipation of a downstream queue formed from vehicles turning onto the main street from an upstream intersection during its main street red interval. Given the turning movements and volumes, the program can calculate queue size for any point in the signal cycle. By summing queue sizes over the duration of a signal cycle, the value for delay in queue (QSUM) is obtained.

Maximal Bandwidth (Time Space Diagram) Method

The objective of this technique is to achieve maximum bandwidths for both directions of travel on an arterial or by giving preference to one direction, maximize the bandwidth in one direction. Little and his associates (60) showed through linear programming methods that in order to achieve equal bandwidths in both directions, half-cycle synchronization of all signals along the street will result in maximal bandwidths. Half-cycle synchronization is accomplished by centering either the green or red interval for each intersection separately on some time reference point (61).

By shifting the offsets from their half-cycle sequence, one can increase the bandwidth in one direction while decreasing it in the other direction. By knowing the predominant traffic flows and points of congestion along an arterial in its present timing, the traffic engineer can adjust the offsets to create a more stable traffic stream. Individual links along the arterial can be assigned special importance and bandwidths increased accordingly.

In order to diagram the traffic signal settings, a two dimensional plot is made of time versus length along the arterial. The signal cycle is drawn along the Y axis (time) at its corresponding location on the X axis. By shifting the beginning of green for each intersection according to half-cycle synchronization or preferential directional movement one can see the bandwidth that will result from a given offset plan.

Application of Timing Plans

In developing signal timing plans by well defined methods and by using intuitive judgement, one is confronted with an infinitude of strategies. As such, this study sought timing plans that were an improvement over the existing system while remaining within the restriction of utilizing existing hardware. Optimality for such a complex system is difficult to define and therefore this report presents the "best choice" strategies from among those studied.

This investigation considered five signal timing plans formulated using the procedures described above. These strategic test conditions are:

Alternative 1: Existing signal timing.

Alternative 2: Existing splits and cycles - delay/difference offsets.

Alternative 3: Webster cycles and splits - delay/difference offsets.

Alternative 4: Webster cycles and splits - half cycle synchronization - time space diagram.

Alternative 5: Webster cycles and splits - preferential street-time space diagram.

Signal timings were developed using these methods, and then analyzed using the UTCS model. Of interest to this study for measuring a system's effectiveness were average speed, stops per vehicle, delay per vehicle-mile, moving time to total trip time ratio, fuel consumption, and carbon monoxide emission.

Using the above alternatives and UTCS as an analysis tool, signal timings for sections of Northwestern Avenue, Grant Street, and State Street were developed. These sections are shown in Figure 1. The findings of this study are based on analyses of these streets and should therefore not be applied elsewhere without due consideration. These findings are divided into two phases.

Phase I concerned itself with concentrating on one street, namely, Northwestern Avenue during the morning peak to investigate the relative values of the aforementioned parameters when cycle lengths were varied over a wide range of typical values. Trade-off curves were developed to see, for example, what effect a reduction in one MOE would have on others. Phase II was intended to develop better signal systems for the streets in West Lafayette, Indiana given the study areas characteristics which include hourly volumes, signal timings, pedestrian flows, and geometric configuration.

CHAPTER 6: IMPLICATIONS OF CYCLE LENGTH ON TRAFFIC FLOW

This phase of the research project involved investigation of traffic flow quality under various common cycle lengths along the arterial of Northwestern Avenue in West Lafayette, Indiana. The map in Figure 1 of Chapter 1 shows the street under consideration. The purpose of this portion of the project was to contribute to the state-of-the-art of signal timing methodology by presenting data and analyses of the effect of signal cycle length on the various measures of effectiveness used to monitor the quality of traffic flow.

It must be realized that effects shown here cannot be applied directly elsewhere without prior investigation. This analysis is applicable to Northwestern Avenue with its unique volumes, turning movements, pedestrian conflicts, turning lanes, and signal phases. To suggest general effects applicable to most anywhere would necessitate a major undertaking with the possibility that specific effects are not attributable to the many timing methodologies. This location-specific study, however, can provide some insight. In combination with the results of past studies, this investigation can become a link in the corroboration of consequences resulting from a signal timing plan. Thus, the observations made here can be used to further substantiate the results of a proposed action.

Development of Timing Alternatives

In order to conduct this task, two general schemes were identified for timing the signals. Within these two schemes, cycles lengths were varied from 50 seconds to 120 seconds in 10 second increments. The first scheme, referred to as Scheme 1, utilized the existing signal timing in operation on Northwestern Avenue. Given the offsets and percentage of green time for each signal, timing plans were calculated for each of the eight cycle lengths studied herein. Originally, the signal timing currently in operation was developed by time-space diagrams and individual phases were adjusted by consideration of approach volumes. Scheme 2 was formulated using Webster's method of determining splits while the delay/difference-of-offset method was applied to these splits to determine the offsets along the arterial. In the subsequent analysis commonalities as well as disparities between the two schemes were noted.

In comparing the differences of traffic flow parameters between the different cycle lengths and the two timing schemes, eight measures of effectiveness were used for this comparison. They are as follows:

1. Average speed in miles per hour;
2. Average number of stops per vehicle trip;
3. Average delay per vehicle-mile in minutes;
4. Cycle failures, defined as the number of times the green phase does not permit the vehicle queued during the previous red phase to clear the approach of the intersection;
5. Average fuel consumption per vehicle in miles per gallon;
6. Average carbon monoxide emitted in grams per mile for a 100 car sample weighted by mix of model years;
7. Average stopped duration per vehicle in seconds;
8. Ratio of delay per vehicle on the minor streets to delay per vehicle on the major street.

Three replicates for each cycle length and scheme were developed using the UTCS model and a different random number seed for each case. Each case was simulated for 10 minutes of real time. Considering the within mean variation generated by the model, it was determined that three subintervals for each case were sufficient to estimate the true mean for the accuracy desired in this segment of the study.

Given the myriad of possibilities of signal timings, the finances available for computer time enters into the determination of the extent of investigation. After careful analysis, it was determined that 10 minutes of simulation time sacrificed little accuracy as opposed to simulating for 15 or 20 minutes. All measures were taken on a per vehicle basis and will therefore be in close agreement with statistics accumulated over a longer or shorter period of time.

General Trade-Off Analysis

Tables 14 and 15 contain the eight measures of effectiveness for each of the eight cycle lengths separated into Schemes 1 and 2, respectively. The values entered in the tables are averages over 3 simulation subintervals of 10 minutes each and are used to develop the following useful observations. Figure 8 illustrates the interrelationship of average speed, emissions, and fuel consumption for the various cycle lengths. Examining the slopes of the three curves, one can see that their steepest region occurs at the cycles from 80 to 100 seconds. Thus a variation in cycle length within this span will result in differences in average speed, fuel consumption and emissions greater than would occur for lower or higher cycle lengths. Specific quantitative analysis cannot be taken from the curves since each scale is unique to each parameter.

Also apparent from the curves in Figure 8 is the fact that decreasing cycle length to 50 or 60 seconds will increase average speed and fuel efficiency while decreasing the amount of carbon monoxide emitted. Although other pollutants (CO_2, HO, NO_x) emitted would tend to increase as the number of stops per vehicle increased, this refinement was undetected in this analysis since only the carbon monoxide levels were derived from regression equations based on fuel consumption. Studies elsewhere have shown that even though a vehicle is getting better fuel efficiency when accelerating than when idling (where mpg=0), carbon monoxide emissions are higher when the vehicle is idling. See Figure 9. As such, longer stopped delays would increase carbon monoxide emissions more so than an increase in the number of stops per vehicle.

Several restraints arise, however, when cycle lengths are reduced to as low as 50 seconds on the arterial studied. Foremost in consideration of cycle length should be the minimum pedestrian crossing times necessary for safe crossings. With left turn and through phases to

Table 14. Measures of Effectiveness for Scheme 1

Cycle Length (sec)	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mile (min)	Cycle*** Failures	Fuel (mpg)	CO* (gpm)	Stop Duration (sec)	Minor/ Major**
50	15.92	1.96	1.68	9.3	10.17	9621	23.74	1.12
60	15.44	1.93	1.81	5	9.97	9792	27.33	1.06
70	15.42	1.85	1.80	1.3	9.96	9804	27.74	0.97
80	15.21	1.78	1.86	1.7	9.88	9878	29.46	1.11
90	15.01	1.77	1.90	0.7	9.79	9952	30.73	1.16
100	14.64	1.76	2.01	1.3	9.64	10090	33.37	1.29
110	14.33	1.72	2.09	0	9.51	10200	35.11	1.24
120	14.24	1.69	2.13	0.3	9.48	10235	36.78	1.45

*Carbon monoxide emitted by a 100 car sample in grams per mile.

**The ratio of minor street delay to major street delay on a per vehicle basis.

***Defined as any cycle during which approach arrivals exceeds the capacity for departures.

Table 15. Measures of Effectiveness for Scheme 2

Cycle Length (sec)	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mile (min)	Cycle*** Failures	Fuel (mpg)	CO* (gpm)	Stop Duration (sec)	Minor/ Major**
50	17.77	1.63	1.29	6	10.93	8945	15.66	1.53
60	16.87	1.71	1.47	7	10.56	9274	19.95	1.74
70	16.12	1.76	1.64	2.7	10.25	9547	24.16	1.39
80	15.71	1.78	1.71	1	10.11	9673	26.20	1.55
90	15.55	1.71	1.77	0	10.01	9756	28.17	1.35
100	14.39	1.76	2.08	0.7	9.54	10177	35.64	1.58
110	15.29	1.65	1.86	0.3	9.91	9851	30.60	1.49
120	14.82	1.67	1.96	0	9.72	10022	33.43	1.55

*Carbon monoxide emitted by a 100 car sample in grams per mile.

**The ratio of minor street delay to major street delay on a per vehicle basis.

***Defined as any cycle during which approach arrivals exceeds the capacity for departures.

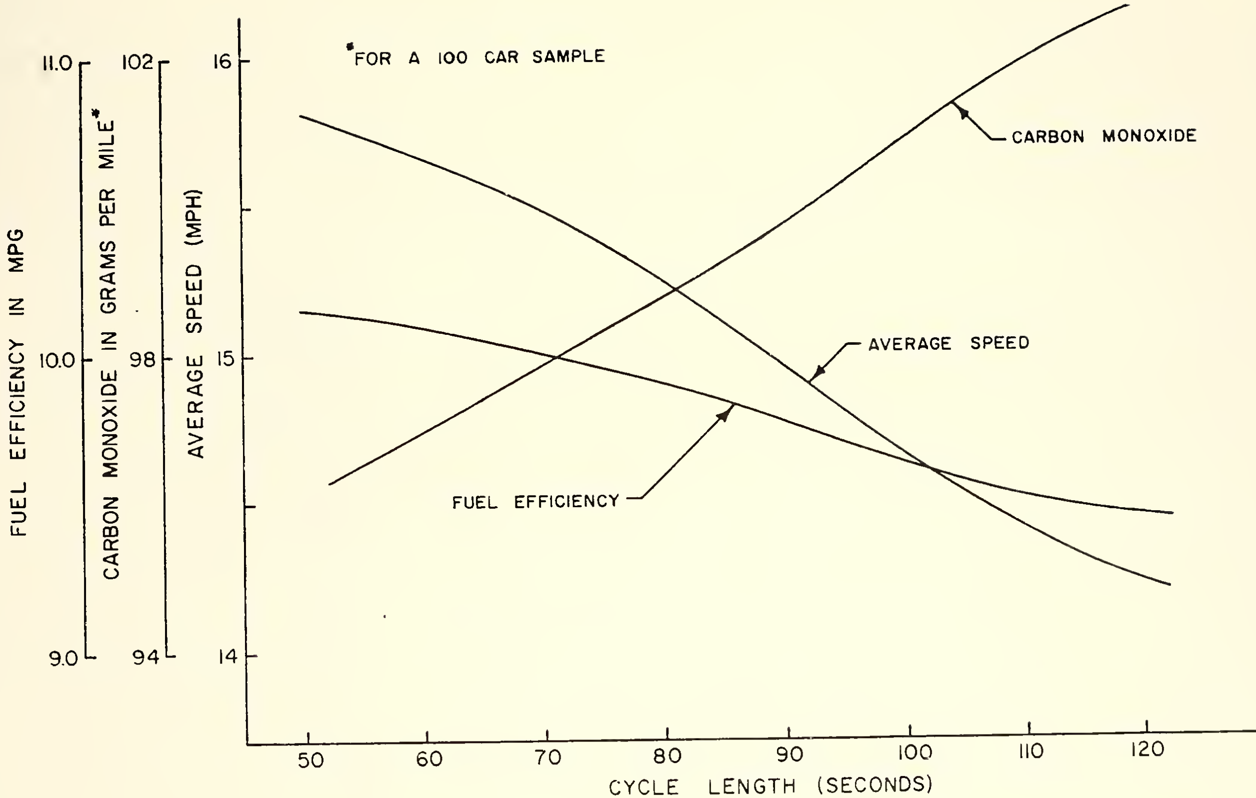

FIGURE 8: INTERRELATIONSHIP OF AVERAGE SPEED, EMISSIONS, AND FUEL CONSUMPTION

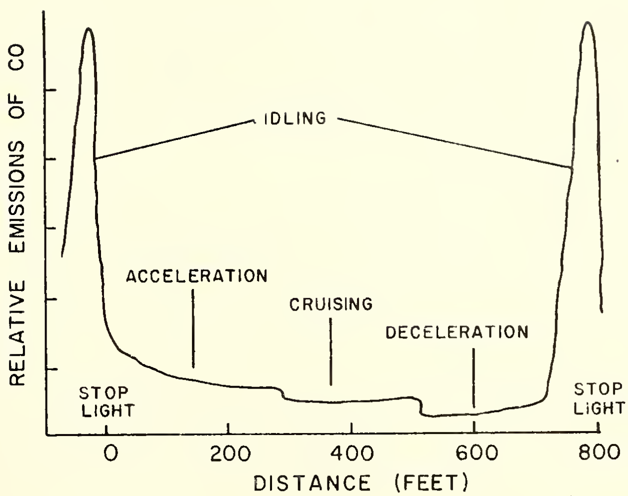

FIGURE 9: RELATIVE EMISSIONS OF CARBON MONOXIDE DURING VEHICLE OPERATION
(SOURCE: REFERENCE 66)

expedite traffic flow, the pedestrian crossing time is further reduced. In areas with heavy pedestrian flows, allotting sufficient crossing time is often the critical variable that determines cycle length.

Figure 10 shows that as the cycle length is decreased to below 70 seconds, the frequency of cycle failures increases abruptly. This is an important factor to consider since an unexpected interruption in the stream will cause an unreleased queue to increase and therefore increase the probability of spillback and the ensuing occurrence of congestion.

In the selection of a cycle length and offset sequence for an arterial, the traffic engineer must weigh the importance of each parameter with respect to another. For example, would the decrease in fuel consumption and CO emission for a short cycle counterbalance the decreased pedestrian safety or increased likelihood of spillback to an upstream intersection? The engineer must continually "trade-off" an improvement in one measure for a resultant detrimental effect on another.

As the delay per vehicle mile increases the resultant carbon monoxide emitted and gasoline consumption increases. This relationship is indicated in Figure 11. Both curves appear to be close to linear relationships and as such a minimization of delay will achieve both fuel efficiency as well as minimization of carbon monoxide emissions. An increase in delay experienced by each vehicle of 12 seconds will cause a 4% reduction in fuel efficiency within the range of values considered.

Analysis of Figures 12 and 13 provides some interesting insight into the elements of delay on the urban arterial. For both Schemes 1 and 2, delay increased as the cycle length was increased. Also plotted on the same axes are stops per vehicle and the stop duration per vehicle. One can see that an increase in the duration of each stop contributes more to the delay experiencedby each vehicle than does the number of stops per vehicle. In Scheme 1 the number of stops per vehicle actually decreases as the delay per vehicle-mile increases. In Scheme 2 delay is increased along with increases in stops per vehicle and stop duration up to a cycle length of 80 seconds. For

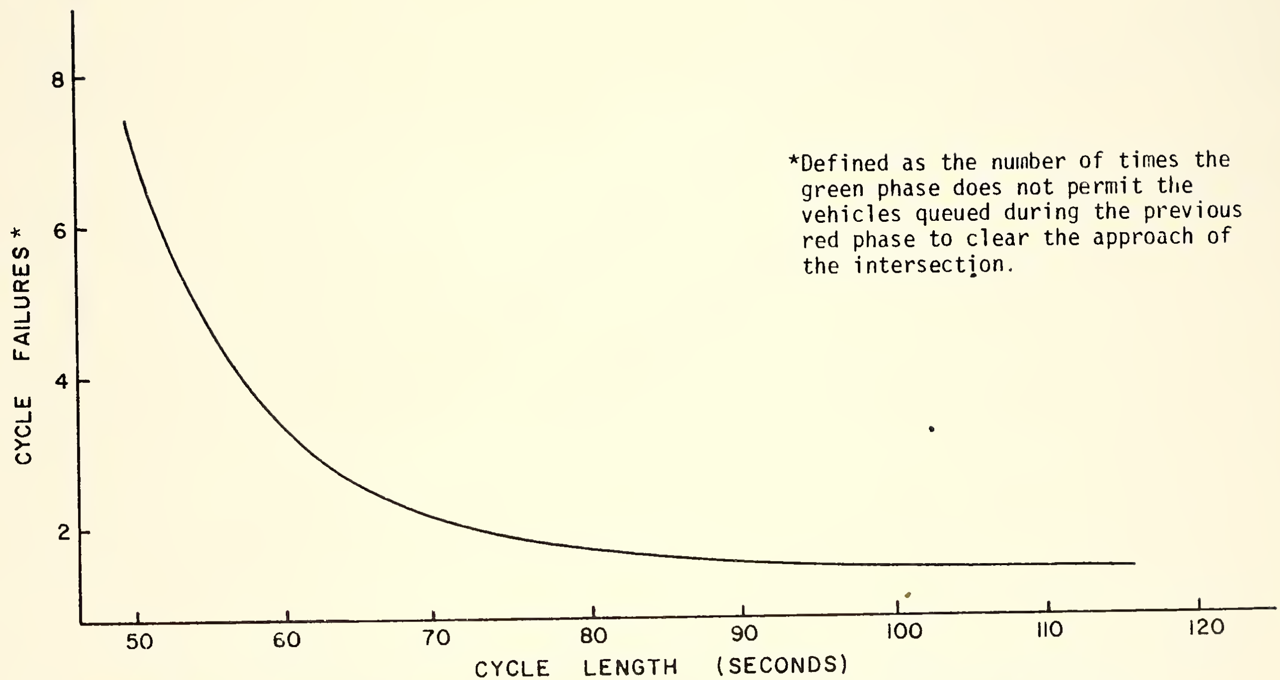

FIGURE 10: EFFECT OF CYCLE LENGTH ON FREQUENCY OF CYCLE FAILURES

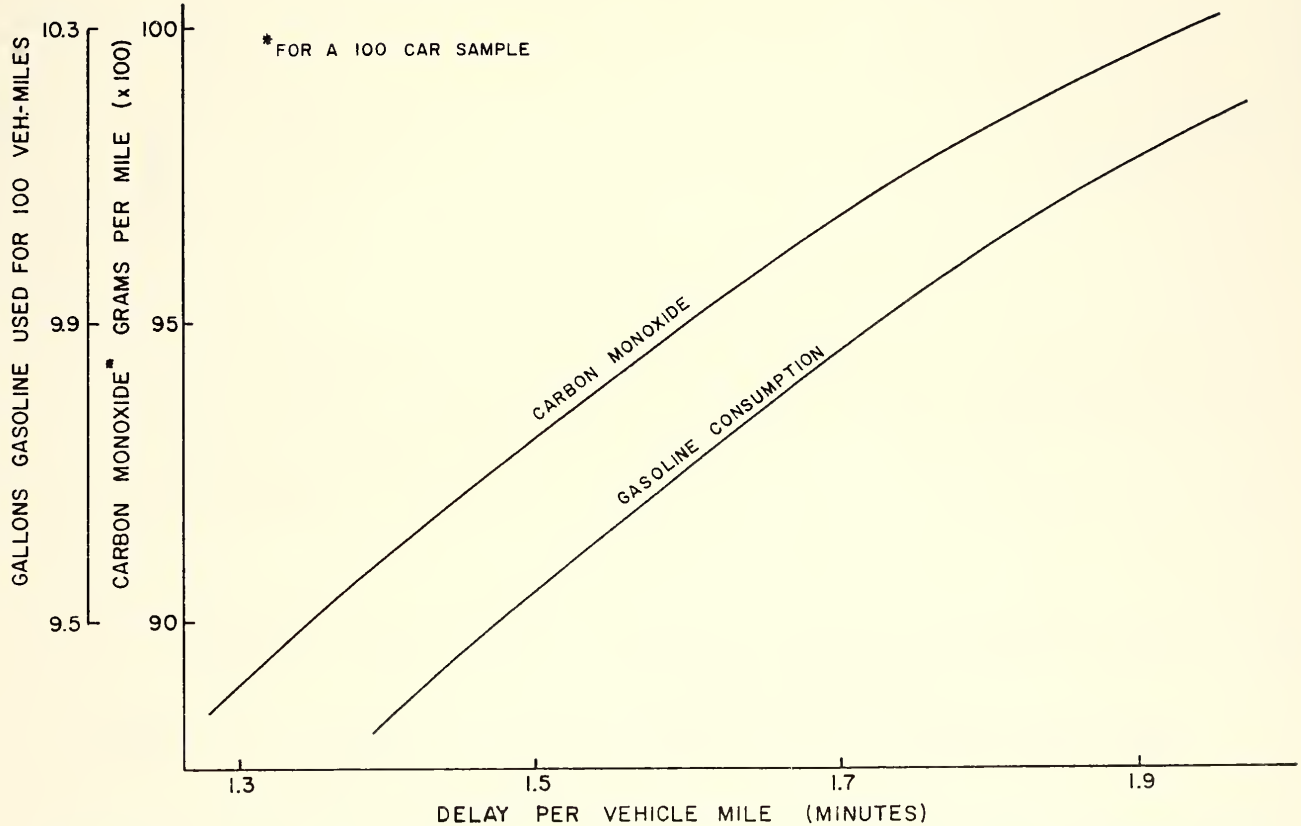

FIGURE II: EFFECT OF DELAY ON FUEL CONSUMPTION AND CO EMISSIONS

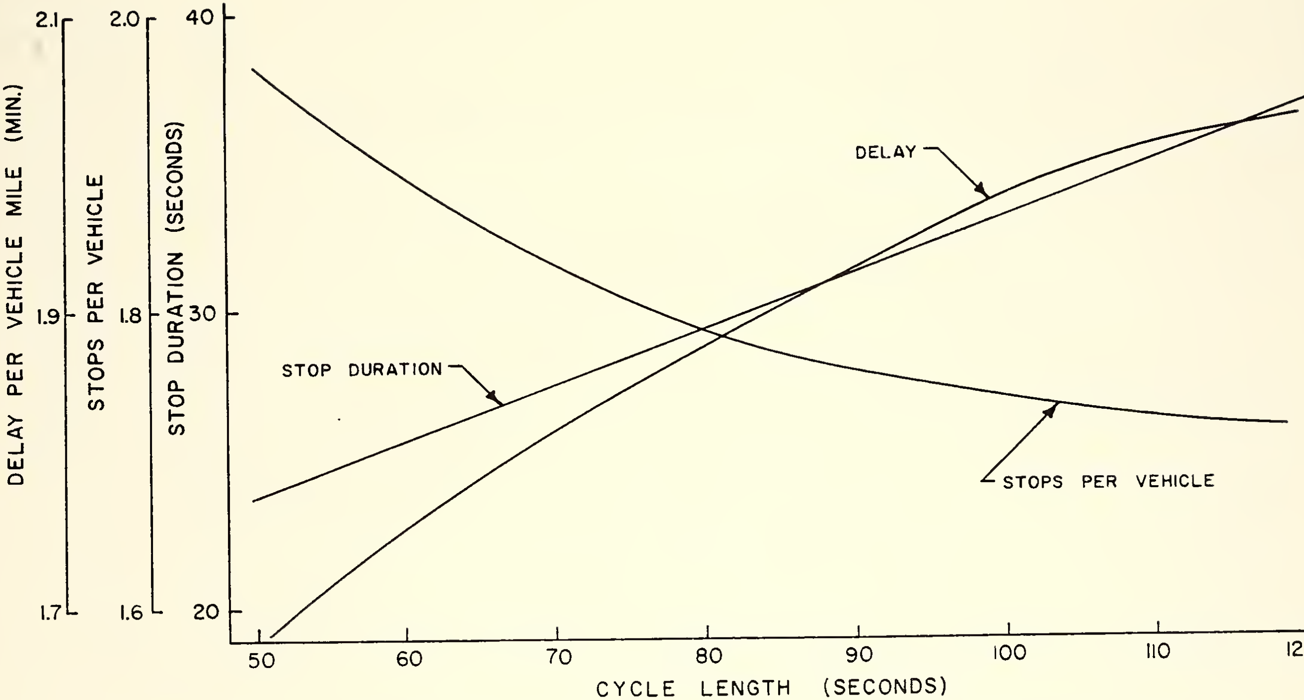

FIGURE 12: EFFECT OF CYCLE LENGTH ON VEHICLE STOPS AND DELAY (SCHEME I)

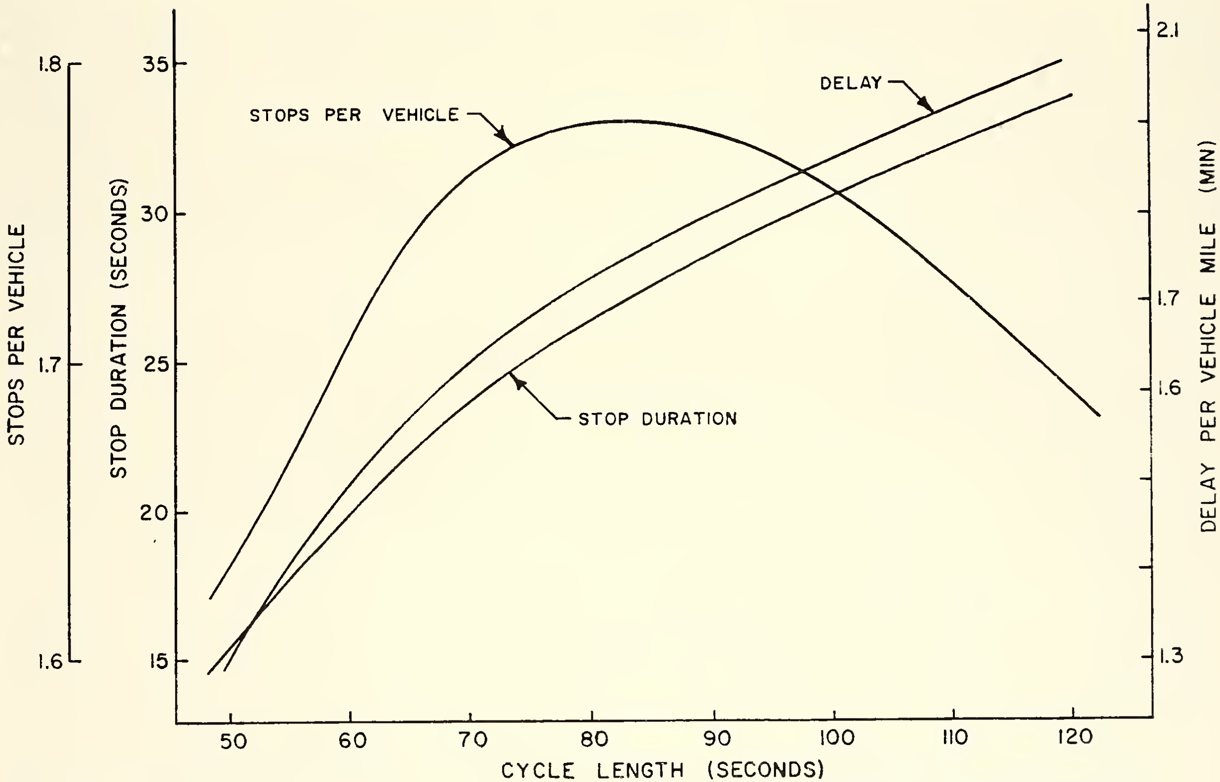

FIGURE 13: EFFECT OF CYCLE LENGTH ON VEHICLE STOPS AND DELAY (SCHEME 2)

cycle lengths longer than 80 seconds, Figure 13 shows that stops per vehicle will decrease as delay and stop duration increase. For both schemes, an increase in cycle length of 10 seconds will result in an increase in stop duration of about 3 seconds per vehicle.

Given two signal timing methods, both cases showed that delay will increase quite proportionally with stop duration. The disparity in the number of stops per vehicle over a range of different cycle lengths given the two schemes is exhibited in Figure 14. Above a cycle length of 80 seconds the two methods produced about the same number of stops per vehicle. When considering cycle lengths in the 50 to 70 second range, about 0.7 stops per vehicle can be avoided by increasing the cycle length by 10 seconds when dealing with Scheme 1. On the other hand a corresponding increase in stops per vehicle would result if the cycle length was increased when Scheme 2 was considered.

Figures 15 and 16 should also be included in the decision-making process of the traffic engineer. Figure 15 shows that the major street volume is experiencing less per vehicle delay with respect to the minor streets for both schemes. Although there is more variation in its trend, Scheme 2 exhibits a consistently higher ratio of minor street to major street delay. For every 30 seconds of delay sustained per vehicle on the major street, a minor street vehicle is delayed about 45 seconds when signals are operated according to Scheme 2. Scheme 1 provides a clear trend towards a more equitable distribution of delay among the major and minor streets as the cycle length is decreased to about 70 seconds. Figure 16 shows that for offsets determined by the delay/difference-of-offset method a higher network average speed is attained for all cycle lengths. Since average speed is a mean weighted by volume, the street with little delay and high volume will greatly influence this mean speed. Such is the case here where the major street in Scheme 2 is experiencing little delay relative to the minor streets.

Given these relationships, one must consider the importance of each parameter with respect to the others. This ranking of parameters according to their importance will vary from community to community.

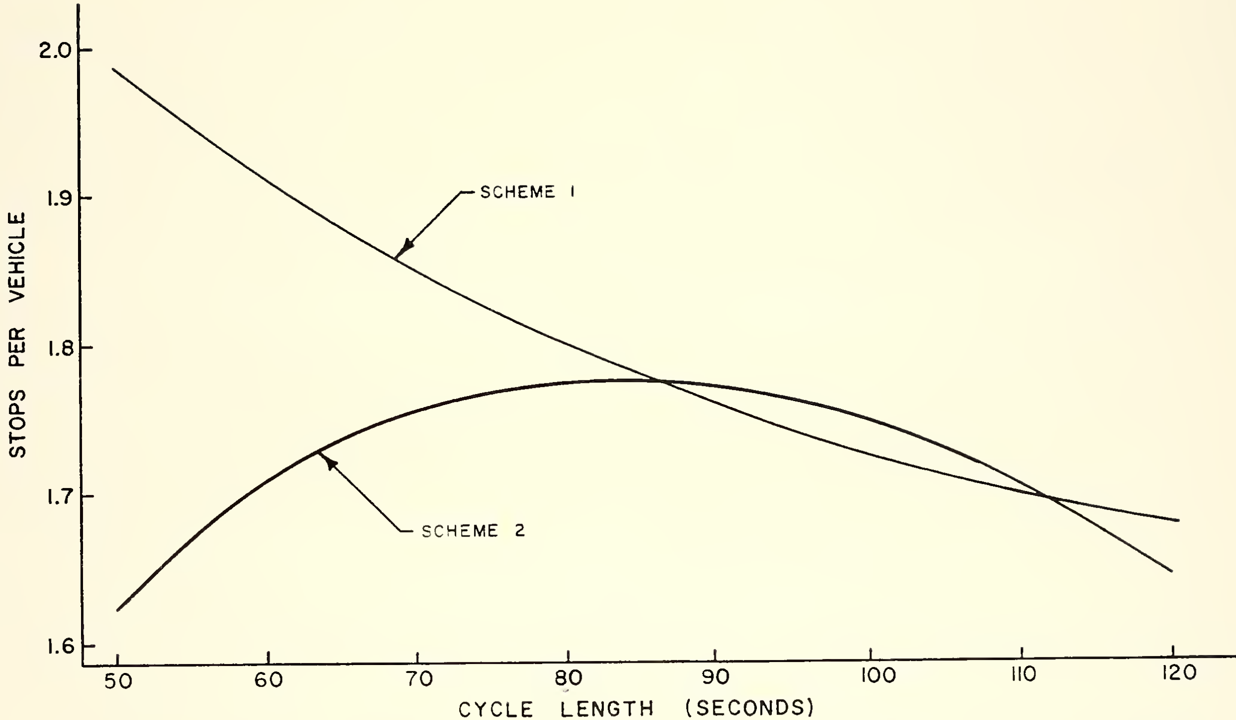

FIGURE 14: EFFECT OF TIMING PLAN ON STOPS PER VEHICLE

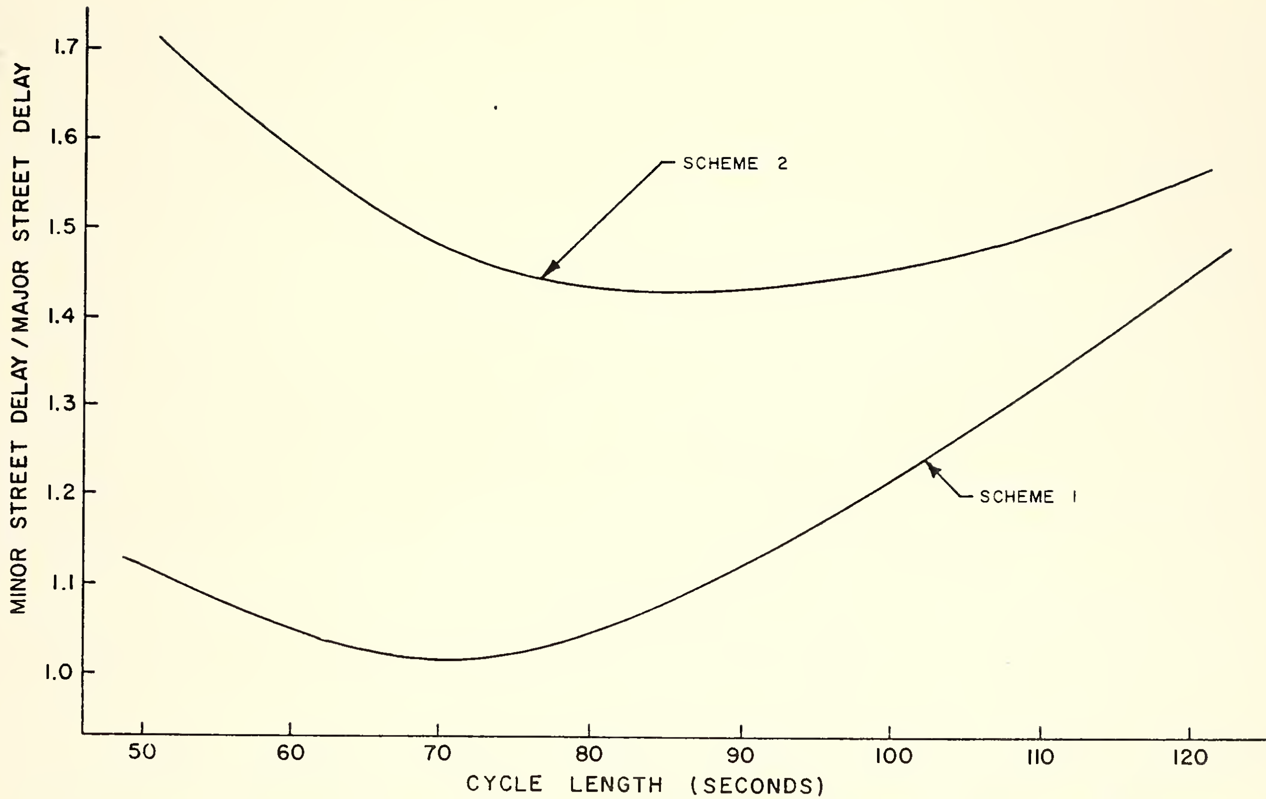

FIGURE 15 : EFFECT OF CYCLE LENGTH AND TIMING PLAN ON MINOR AND MAJOR STREET DELAY

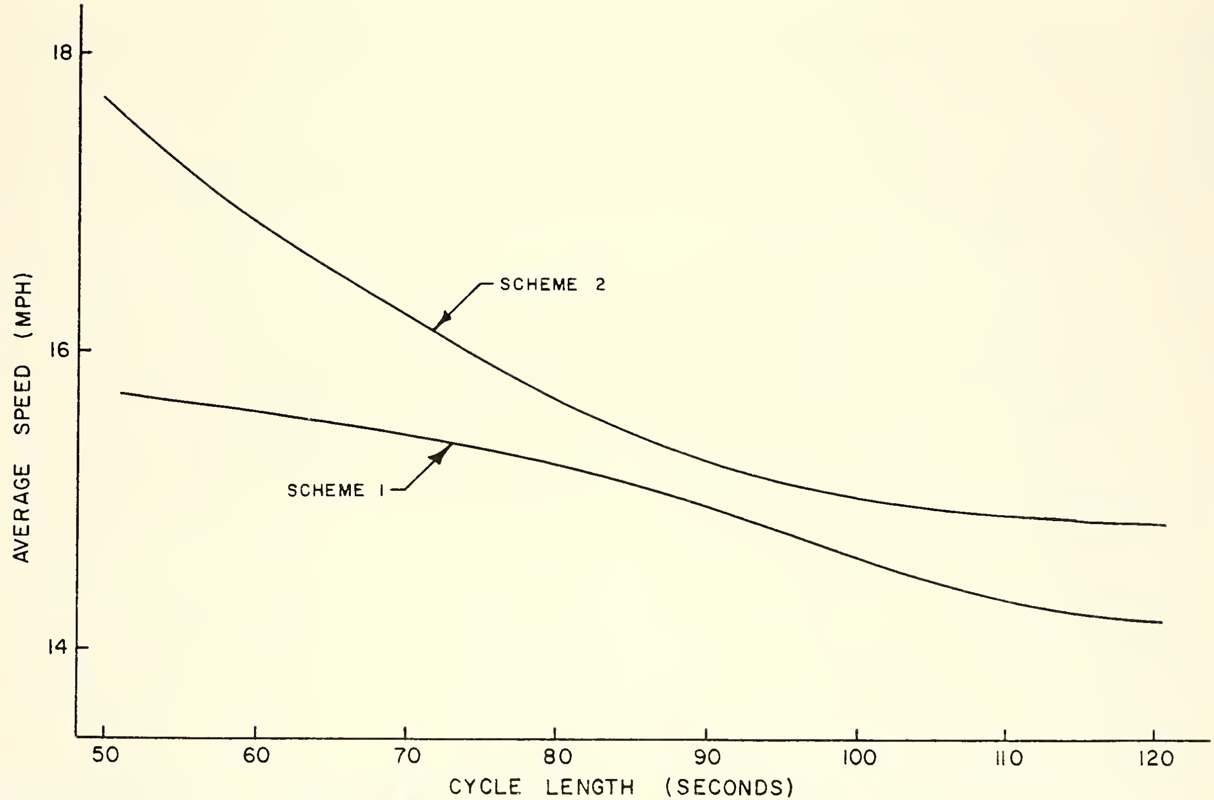

FIGURE 16 : COMPARISON OF EFFECT OF CYCLE LENGTH AND TIMING PLAN ON AVERAGE SPEED

For example, is it desirable to favor the major street at the expense of all the minor streets on this particular arterial?

Also included in the evaluation phase is the question of safety associated with each alternative. Considering only Scheme 1, it will provide higher average speed, better fuel efficiency, lower CO emissions, and less delay when operated with the 50 second cycle; however, the number of stops per vehicle is greatest. With an increase in stops per vehicle, the acceleration noise of the arterial increases denoting a higher probability of accident occurrence. Wear and tear on vehicles from stopping and starting would be a factor to consider in the selection of signal operation. The aggressiveness of drivers and their attentiveness to driving would also affect the flow of traffic in a frequent stop situation. Furthermore, as previously mentioned, the precipitous increase in cycle failures for short cycle lengths (<70 seconds) will also serve to negate the positive effects of cycle lengths that are of short duration.

Using the volumes, pedestrian traffic, turning movements, and geometric configuration determined from the test area, one can see that, generally, as the signal cycle length decreased - while still allowing ample pedestrian crossing time and avoiding frequent cycle failures - additional improvements to the flow of traffic would be achieved under both schemes. Clearly, increasing the cycle length by 15 seconds or more above that which is necessary would tend to annul any improvements on the arterial.

Also shown in comparing Scheme 1 with Scheme 2 over the range of cycle lengths studied was that the application of some systematic analysis to the development of a control strategy would very likely produce a better index of performance along the arterial - considering energy, pollution, and delay. Figure 16 indicates the disparity in the average speed for Schemes 1 and 2; Scheme 1 refers to the existing system while Scheme 2 included a systematic technique in developing the signal timing plans. The higher average speed realized under Scheme 2 indicates the usefulness of applying some proven method in determining signal operation. In this case, the delay/difference program has led to a system operation with consistently higher average speed over all cycle lengths.

In comparing Scheme 1 and Scheme 2, it is seen that no clear-cut method or mathematical expression can be used to determine the cycle length that would create the most desirable situation in all locations. Some definitive community policy must be used in the decision-making process as to what measures of effectiveness are important and clearly outweigh less attractive parameters. The setting of the arterial in the urban street system will also contribute to the selection of a control strategy as will the distribution of vehicle arrivals throughout the hour. Disproportionate amounts of turning movements will also affect the timing plans and should be recognized in applying the above mentioned techniques.

If some quantifiable weights were assigned to each of the measures of effectiveness one could proceed with a mathematically rigorous analysis. Once coefficients are determined for each of the parameters, mathematical programming techniques to optimize a specific objective function may be applied. Constraints attributed to signal offsets and to each of the cycle lengths would also have to be specified. This technique would then arrive at an optimal solution for the input coefficients provided.

The foregoing presentation has served to provide some insight into the application of signal timing to a system of signals along an arterial. Fuel consumption and environmental quality should become a major consideration in the design of systems which affect the movement of traffic. While the results herein are generally location specific, trends that were observed can provide more general implications. On-going research can further substantiate these observations and thus a seminal basis for traffic research was initiated. Furthermore, simulation models (particularly UTCS-1) are being refined to directly estimate fuel and emission parameters in the quest for more prevalent usage and acceptance in determining the effects of a proposed alternative before action is undertaken.

CHAPTER 7: EFFECTS OF SIGNAL TIMINGS FOR THE STUDY AREA

The application of a traffic simulation model to determine the merits of signal timing plans would avoid much of the trial and error inherent in conventional formulation. This phase of the research sought a two-fold outcome. The first and foremost desired effect was one of demonstrating and promoting the use of the UTCS model as a useful tool for the selection of operational modes on arterials. The second objective was to present five alternative signal timing plans for the study area with their respective measures of effectiveness so that the effects of each could be examined in a system analytic framework. Cycles, splits, and offsets would be provided to permit actual implementation of the timing plans if desired.

Whereas the installation of a particular timing plan may appear to be better than a previous one based on intuitive judgement, without simulation and its detailed output, concrete evidence is often difficult to ascertain short of time consuming studies based on large amounts of data.

Study Area Description

Development of signal timings was based on the study area as shown in Figure 1. The three major arterials considered were Northwestern Avenue, State Street, and Grant Street. Plans were developed for the AM peak, Off peak, and PM peak periods. Input data to the UTCS model was based on volume counts with turning movements from previous years. Current signal timing splits and offsets supplied by the Indiana State Highway Commission and from actual observation were used for modeling the base case.* Other input data necessary for simulating the street

---

*All signals except those at the Grant-Wood and Grant-North intersections are state owned. The latter are property of the City of West Lafayette.

system was collected in the field. Also, periodic observations of the existing system to note specific problems with traffic movement during the peak periods assisted in the development of new signal timings as well as for verification of correct data input and reasonable output.

Each of the signals in the study area are operated by 3-dial controllers. Each of the three dials is capable of containing a complete cycle length with splits for that particular signal. Cycle lengths from 30 to 130 second cycles are possible. By switching from one dial to another during the day, one can implement different signal timings to accommodate the variable traffic flow which is usually divided into three periods. It is assumed that the volumes will occur at approximately the same time during each day. In addition to the keys on the dials which specify the intervals for each phase, an offset key may be used to establish a desired offset between intersections along the arterial. A master controller would be used to synchronize the signals according to a user specified offset plan. This master source would stop an individual signal's dial until the master and cycle unit are at the desired point in a cycle with respect to the master reference (62). Signals along Northwestern Avenue are interconnected as are the signals along State Street. Signals along Grant Street are not interconnected and as such operate independently of each other. No interconnection is provided between the State Street arterial and the Northwestern Avenue arterial. Because of the lack of interconnection, the three streets were considered separately of each other. Timing plans developed considered each arterial as a distinct entity. Easier calculations and timing formulations were a result of analysis on an arterial basis.

Simulation of Existing System

After the data was collected and collated, the streets were divided into links and nodes for coding the systems in the manner necessary for model input. This link and node representation for each of the three main streets as well as the volumes entering the street systems are presented in Appendix C. The volumes given are for the AM peak (8:00 a.m.), off peak (10:00 a.m.), and PM peak (5:00 p.m.) periods. These vehicle flows formed the basis for the subsequent development of timing alternatives and therefore specific timings are heavily dependent on these specific volumes. Further refinements may be needed in the timing plans based on the level to which the volumes vary in the future. Given the data required to model the arterials, initial computer simulations were run for the three streets for each of the three periods.

Initial runs revealed some data preparation errors. The UTCS-1 program provides excellent diagnostics which greatly facilitate debugging of the input data. The errors encountered were mainly due to incorrectly punched data cards. Once the errors were corrected sufficiently to allow execution of the program, output data describing the arterial was closely scrutinized for reasonableness before proceeding further. A few errors, including a wrong interval for a green phase were detected and corrected. Familiarity with the system under consideration was instrumental in this error detection. More subtle errors can only be detected by close examination of the input data deck. (Part of the UTCS-1 output is the input data deck as provided by the user. See Figure A5, Appendix A.)

Generation of Timing Plans

Timing plans were generated using the methods described in Chapter 5. Analyses and comparisons with the existing systems was conducted using UTCS as the medium for quantifying the measures of effectiveness for each alternative signal timing plan. Impacts were noted in terms of average speed, stops per vehicle, delay per vehicle-mile, fuel consumption, carbon monoxide emission, stop delay per vehicle, and the ratio of delay on the minor street to the delay on the major street on a per vehicle basis.

Five timing plans were considered in the analysis of traffic flow on Northwestern Avenue and on State Street. Because of the lack of signal interconnection along Grant Street a different strategy was used. For the first two streets the timing plans developed were based on the following methods for determination of the cycle length, splits and offset from signal to signal along the arterial:

Timing Plan 1: Existing splits and cycles - Existing offsets;

Timing Plan 2: Existing splits and cycles - Delay/difference offsets;

Timing Plan 3: Webster splits and cycles - Delay/difference offsets;

Timing Plan 4: Webster splits and cycles - Half cycle synchronization offsets (from time-space diagram);

Timing Plan 5: Webster splits and cycles - Preferential street treatment (from time-space diagram).

Grant Street was timed using Webster'soptimization procedure of timing signals individually since no interconnection of signals exists along Grant Street. Refinements were made in the timings in efforts to further improve traffic flow.

In all, 38 different timing plans were included in this analysis. Considerable computer time was necessary in developing and simulating each of these plans. Given a fixed expenditure limit and a desired minimum level of accuracy, it was determined that three random replications of each plan would sufficiently approximate the true population mean. A different seed for the random number generator was used in each case. This random number generator serves to produce variations in flow and random aspects of drivers typical of an urban environment.

In all cases, due consideration was given to pedestrian movements. Signal intervals were kept sufficiently long to allow for the safe crossing of pedestrians. Also, since UTCS can incorporate pedestrian interference of the traffic stream, this option was used.

An estimate of the number of vehicle trips and vehicle miles traveled during the simulated ten minute periods for each street during the different periods of the day is shown in Table 16. These figures are used in the subsequent determination of impacts of the various timing plans. Since these figures will vary depending upon the efficacy of a specific plan, the estimates are only intended as

Table 16. Vehicle Trips and Vehicle Miles for the Three Arterial Sections*

Street (Period)	Vehicle Trips	Vehicle Miles
Northwestern (AM)	450	200
Northwestern (Off)	330	150
Northwestern (PM)	550	230
State (AM)	450	250
State (Off)	410	230
State (PM)	625	340
Grant (AM)	550	200
Grant (Off)	400	150
Grant (PM)	775	275

*For 10 minute periods.

guidelines from which some indication of the magnitude of time-savings or fuel savings may be achieved.

The ensuing analysis is broken down by arterial with the expected impacts noted for the various signal timing plans. Signal settings for all arterials studied herein are presented in Appendix D. Offsets along the arterial, where applicable, are also tabulated in the appendix.

Northwestern Avenue Signal Timing Analysis

All of the subsequent tables showing measures of effectiveness, list the mean simulated measures taken from three simulation subintervals of 10 minutes each. Also, the MOE's cited are as follows: average speed in miles per hour, the number of stops per vehicle trip, the delay per vehicle mile in minutes, the rate of fuel consumption in miles per gallon, the carbon monoxide emissions (CO) for a 100 car sample typical of the model year vehicle mix for Indiana in grams per mile, the stop delay per vehicle trip in seconds, and the ratio of the delay experienced by motorists on the minor street to the delay experienced by motorists on the major street on a per vehicle basis. Simulation was run on a CDC 6500 computer. A real-time to simulation time ratio of about 8 to 1 was realized when simulating Northwestern Avenue.

The MOE's of the various timing plans for the AM peak period along Northwestern Avenue are listed in Table 17. While any one MOE will usually provide a fair indication of a timing plan's relative merits, this is not always the case. As seen in the previous chapter, the number of stops per vehicle trip may actually decrease as the delay per vehicle mile increases. In this light one must consider the total effects of a particular scheme to make an adequate judgement.

Plan 1, which represents the existing system, exhibits a network average speed of 15.01 miles per hour. All of the other plans served to better each MOE except the minor street delay to major street delay ratio. An improved value of this ratio is dependent upon the discretion of the engineer. (A ratio value of 1 would indicate that a vehicle is

Table 17. Northwestern Avenue AM Peak Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
1	15.01	1.77	1.90	9.79	9952	30.73	1.16
2	15.37	1.71	1.81	9.94	9821	29.13	0.96
3	16.12	1.76	1.64	10.25	9547	24.16	1.39
4	16.40	1.69	1.58	10.37	9443	23.14	1.50
5	16.60	1.67	1.51	10.45	9372	21.94	2.05

*Carbon monoxide emission for a 100 car sample in grams per mile.

experiencing the same delay regardless of the nature of the link on which it is traveling.) Implementation of Plan 5 would increase the average speed by 11% while reducing emissions of carbon monoxide by 580 grams per mile for a 100 car sample. In addition, a savings of time of about 8 hours would be realized during the 1-hour morning peak period. However, with Plan 5 in operation, the minor street vehicles would experience approximately twice the delay experienced by major street vehicles. A more equitable distribution of delay would be created by implementation of Plan 4 which was formulated using Webster splits and half-cycle synchronization. Some measure of safety may be ascribed to the system in that 45 fewer stops would occur during 10 minutes when operation is according to Plan 5 as opposed to Plan 1. Acceleration noise - a measure of the smoothness of traffic flow - would decrease as the number of stops decreases, everything else held constant. A reduction in acceleration noise would serve to indicate a safer driving environment.

Table 18 records the MOE's for the off peak traffic condition. Volumes for input were taken at 10 a.m. It is assumed that these conditions remain approximately the same throughout the day except during noon lunch hours and the evening peak period. While an improvement in average speed is gained over the morning peak with the existing settings, a further improvement is possible by implementing Plan 3. Even though the minor streets experience more delay with the latter plan, all of the other MOE's are significantly improved. This plan was formulated using 50 second cycles and the delay/difference-of-offset method described in Appendix B. Included in its benefits are an 11% increase in average speed, a 38% reduction in stop delay per vehicle and an increase of 8% in fuel consumption efficiency. The number of stops per vehicle trip was reduced by 17%, indicating a more stable traffic stream. Plan 2, which applied the delay/difference offset technique to the existing cycle and splits, did not improve the level of service of the arterial. In every instance, the application served to deteriorate the MOE's. Only when an offset plan was developed in conjunction with Webster's determination of splits and common cycle length was an improvement shown.

Table 18. Northwestern Avenue Off Peak Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
1	17.17	1.70	1.44	10.68	9163	19.98	1.06
2	16.56	1.76	1.58	10.43	9386	23.21	1.17
3	19.13	1.41	1.08	11.49	8447	12.31	1.25
4	18.71	1.45	1.15	11.31	8601	13.76	1.10
5	18.64	1.59	1.15	11.29	8627	12.95	1.53

*Carbon monoxide emission for a 100 car sample in grams per mile.

A substantial reduction in the delay per vehicle-mile is shown in Table 19 under Plan 3. Under the afternoon peak traffic conditions the timing scheme arrived at by the delay/difference method once again produced a very acceptable mode of operation of signals. Each vehicle's stop delay was reduced, on average, 9 seconds, while the number of stops for a typical one hour afternoon peak period was reduced by about 30. However a larger percentage of the delay is attributed to the major street than to the minor street when either Plan 3, 4, or 5 is considered. Because of the small increase of average speed attained in the improvement, only about 1 gallon of fuel per 10 minute period would be saved when Plan 3 is implemented over the existing signal timings.

Given that approximately 15% of the vehicle-miles are traveled during a peak hour*, daily fuel savings may be calculated. If Plans 5, 3, and 3 were instituted for the morning, off, and evening peaks, respectively, a daily fuel savings of about 60 gallons would be achieved. Similar reductions in carbon monoxide emissions would be realized. Also, a daily reduction of more than 4000 vehicle stops could be attained. It is therefore demonstrated that a significant improvement in urban transportation efficiency can be achieved through proper traffic management schemes at signalized intersections and simulation studies can be effectively utilized to establish such management schemes.

State Street Signal Timing Analysis

Because of the larger size of the system simulated and the increase in the number of vehicles traversing it, the real-time to simulation time ratio for the State Street arterial was reduced to 5 to 1. A longer initialization time was also required to enable the system to reach equilibrium before statistics could be accumulated.

---

*Generally about 12-15% of the total number of trips occur along an arterial during a peak hour. Therefore, an estimate of 15% of the total vehicle miles traveled occurring during a peak hour is not unreasonable.

Table 19. Northwestern Avenue PM Peak Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
1	15.16	1.72	1.88	9.86	9868	29.41	1.05
2	15.47	1.67	1.80	9.98	9783	27.72	1.09
3	16.52	1.67	1.57	10.42	9412	21.57	0.92
4	15.38	1.94	1.86	9.93	9837	27.63	0.79
5	16.49	1.76	1.66	10.24	9553	23.35	0.83

*Carbon monoxide emission for a 100 car sample in grams per mile.

Table 20 shows the MOE's for the five timing plans for State Street developed for the AM peak period. The existing system is characterized by an average speed of only 10.18 miles per hour and a delay per vehicle-mile of 3.90 minutes. Additionally for every 20 seconds of delay per minor street vehicle, the major street per vehicle average delay is 53 seconds; an indication of too much minor street preference is apparent. A more equitable distribution of major-minor street delay was gained by using the delay/difference offset technique coupled with the Webster optimal cycle (Plan 3). A 62% increase in average speed and a 66% reduction of stop delay per vehicle trip was noted. The half-cycle synchronization method of offset determination produced slightly better mean MOE's although not statistically significant at a level of significance of α = 0.05.

The primary difference between Plan 3 and Plan 4 for the morning peak period lies in the ratios of minor street delay to major street delay. Plan 3 provides for 1.24 minutes of delay per vehicle on the minor street for every 1.00 minutes of delay on the major street; Plan 4 gives more preference to the major street in that its ratio is 1.47.

About 50 fewer gallons of fuel will be consumed during each morning peak hour when the system is operated in accordance with either Plan 3 or Plan 4. Furthermore, about 35 fewer kilograms of carbon monoxide will be emitted per morning peak hour under the revised system. These statistics, when converted into annual amounts, indicate a significant improvement in traffic control efficiency.

It should be noted that even though the average speed, stop delay per vehicle, and fuel consumption efficiency were improved substantially in Plans 3 and 4 as compared with Plan 1, the number of stops per vehicle trip remained somewhat constant throughout the three plans. The improvement in the level of service can largely be attributed to the large reduction in the duration of idling time.

Although improvements were not as great under the developed plans for the off peak period, the betterments are certainly worthy of consideration. The most significant improvement was achieved through application of half-cycle synchronization (Plan 4) as shown in Table

Table 20. State Street AM Peak Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
1	10.18	1.78	3.90	7.81	11717	93.64	0.38
2	10.38	1.87	3.82	7.89	11643	91.41	0.40
3	16.50	1.83	1.60	10.40	9411	31.88	1.24
4	16.58	1.80	1.56	10.44	9377	31.17	1.47
5	15.57	1.86	1.81	10.03	9748	38.32	0.85

*Carbon monoxide emission for a 100 car sample in grams per mile.

21. Space mean speed increased from 16.12 mph to 17.46 while the number of stops per vehicle trip remained fairly constant. Delay per vehicle-mile for a typical off peak hour amounts to 18 seconds more under the existing system than with Plan 4 in operation. Delay is also more evenly distributed with the proposed signal timing scheme (Plan 4), as the ratio is closer to unity.

Here also, the application of the delay/difference-of-offset method to the existing cycles and splits actually decrease the efficacy of the control. All of the MOE's worsened. The time-space diagrammatic approach to the offset determination appeared to produce the better signal timing plans.

As previously pointed out in the description of the UTCS model, simulation results during periods of light traffic flow are not quite as accurate as are results from well-disciplined traffic flow. This point is again underscored so as not to misguide the potential user. Small improvements in traffic flow as shown by simulation should be more carefully studied before implementation if an alternative action is undertaken. This is particularly important if a high capital cost is associated with the proposal under study. Even for periods of dense traffic flows, the user must continually verify the reasonableness of the output. The concept of feeding data into a "black box" must be avoided as input errors would then easily go undetected.

Table 22 shows the MOE's for State Street's afternoon peak periods. As can be seen, insignificant improvements were attained by all of the developed timing plans. This indicates that the existing timing plan is already an efficient scheme. Further manipulation of the signal settings and offsets produced no better results. The only differences notable among the schemes are the minor street delay to major street delay ratios. A design procedure incorporating simulation will not always lead to a better plan; this is not the purpose in the use of simulation. The aid of a rapid medium to arrive at a signal timing which can be analytically compared with other alternatives is the true merit of a simulation model.

Table 21. State Street Off Peak Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
1	16.12	1.75	1.68	10.25	9546	36.35	1.92
2	15.58	1.90	1.86	10.03	9744	40.82	1.47
3	16.18	1.91	1.65	10.28	9525	34.13	1.30
4	17.46	1.72	b.38	10.81	9056	27.12	1.78
5	17.04	1.78	1.48	10.63	9210	30.22	1.71

*Carbon monoxide emission for a 100 car sample in grams per mile.

Table 22. State Street PM Peak Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
1	15.34	1.89	1.87	9.93	9830	38.45	1.59
2	14.18	1.91	2.18	9.45	10256	47.91	1.19
3	14.93	1.88	1.97	9.76	9980	42.01	1.57
4	14.73	1.82	2.03	9.68	10051	44.04	1.75
5	15.40	1.83	1.85	9.96	9808	38.33	2.20

*Carbon monoxide emission for a 100 car sample in grams per mile.

Grant Street Signal Timing Analysis

Table 23 lists the timing plans for Grant Street. The MOE's for each of the three periods of the day are shown in this table. About 6 seconds of real time were simulated with 1 second of central processing time. Since the signals along Grant Street are not interconnected to permit synchronization, the offset determination techniques were not used. Instead, only the Webster method of calculating splits and cycles was applied in seeking a better system of operation based on the MOE's heretofore considered. Plan 1 for each of the periods represents the existing signal timings. The second plan for each peak was derived by Webster's equations. A third attempt was undertaken for the morning and afternoon peak periods by adjusting the splits using the link-by-link MOE's as guidelines.

Spillback - the effect caused by an upstream queue interfering with a downstream signal - occurs during the morning peak hour on Grant Street from State Street to North Street (link 50, 60) under the existing signal timing. Simulation of Plan 2 - Webster's splits - showed an increase in average speed from 11.15 mph to 13.75, a 23% increase. A 40% decrease in the stop delay per vehicle trip was accompanied by no change in the number of stops per vehicle. In addition, the use of Plan 2 showed no spillback effect. However a further attempt at refining Plan 2 resulted in a negation of the improvement; spillback again occurred.

A probable reason for the low ratio of minor street delay to major street delay is that although State Street and Northwestern Avenue are major arterials, the consideration of Grant Street in this case classifies both State and Northwestern as minor streets. Consequently all timing plans for the three segments of the day result in ratios less than unity.

Efforts to produce better flow along Grant Street during the light volume mid-day traffic and during the afternoon peak period were not as successful. The mean speed for the PM peak was raised from 14.77 to 15.10 mph, an insignificant increase. The proportion of minor street delay to major street delay was changed from 0.48 to 0.68.

Table 23. Grant Street Measures of Effectiveness

Timing Plan	Average Speed (mph)	Stops/ Vehicle	Delay/ Veh-Mi (min)	Fuel (mpg)	CO* (gpm)	Stop Delay/ Vehicle (sec)	Minor Dly / Major Dly (per veh)
AM Peak							
1**	11.15	1.31	3.48	8.21	11363	58.50	0.28
2	13.75	1.31	2.35	9.27	10413	35.32	0.33
3**	11.52	1.37	3.22	8.36	11226	52.57	0.63
Off Peak							
1	17.15	1.10	1.45	10.68	9170	18.20	0.63
2	17.19	1.14	1.46	10.69	9157	17.92	0.81
PM Peak							
1	14.77	1.30	2.01	9.69	10041	27.81	0.48
2	15.10	1.33	1.91	9.83	9919	25.66	0.68
3	15.00	1.34	1.96	9.79	9956	26.34	0.64

*Carbon monoxide emission for a 100 car sample in grams per mile.

**Occurrence of spillback during simulated time interval.

Area-Wide Improvements

Given the system of signals for the West Lafayette Study Area, a systematic approach was initiated for seeking improvements to the flow of traffic through these signals. Quantification of the MOE's for a specific alternative timing plan was made possible with the UTCS-1 simulation model. Overall, a real-time to simulation time ratio of about 6.5 to 1 was achieved for each of the arterials simulated.

Implementation of each of the improved signal timing plans for the 3 periods of the day would result in considerable savings of travel time, fuel, and environment deterioration. No doubt, further improvements could be gained if a coordinated system of signals were to be installed on Grant Street. Based on calculations from the simulation output for the study area the following estimates were made of potential benefits in going from the existing system to the improved timing plans: a daily reduction in delay of 173 hours would be experienced by the new system; approximately 150 gallons of fuel would be saved daily as well as a reduction of more than 100 kilograms of carbon monoxide emitted. These benefits are possible by simply resetting the dials in the controllers of each signal. No capital cost would be involved.

Further study would be beneficial in the determination of what hours are to be designated for each of the periods for which signal timings were developed, i.e., when each of the controllers dials should be in operation. Also, periodic volume counts must be taken to update signal timings. If personnel conditions permit, seasonal variations in traffic flow should be accounted for by refinements in the signal settings. An areawide comprehensive perspective must be used in trying to predict fluctuations or disturbances to the volumes for which the signals are set. For example, a closed bridge outside of one's jurisdiction may well affect the local flow of traffic including system turning movements.

Of the signal offset methods applied to the study area arterials, the delay-offset difference consistently gave good results when used with the Webster equations. Moreover, when the Webster technique for determination of cycles and splits was combined with the offset methods improvements in the operation were realized. In the few cases where improvement was not made by these techniques, signal timings - often empirically derived - had probably been implemented prior to this study; much of the inefficiency in handling the traffic stream had been reduced by these timings.

The Webster method requires only hand calculations based on easily accessible data including critical lane volumes and queue discharge rates. The delay/difference computer program supplied in Appendix B is relatively easy to use and provides effective timing plans. As seen from the output of the delay/difference program, one can give priority to directional volumes or merely provide more emphasis for a specific direction. Also, since the program works with two links at a time, a network of offsets may be "built" to handle a grid type street network.

Given the information used in determining the signal timings along the three arterial streets studied, the author recommends the following timing plans for each street and period of the day:

Northwestern Avenue	Timing Plan
AM	4
Off	3
PM	3
State Street	
AM	3
Off	1
PM	1
Grant Street	
AM	2
Off	1
PM	1

The specific timings can be ascertained by referring to Appendix D where the splits and offsets are given.

CHAPTER 8: EFFECT OF ELIMINATION OF A LEFT TURN PHASE

A further evaluation was carried out utilizing the simulation model as an analysis tool. In many cases an added phase on a signal is unquestionably needed during only a small portion of the day. This is oftentimes caused by an industry releasing its workers at a certain time each day. Notwithstanding political pressure and a certain degree of public relations, the traffic engineer may want to assess the impact of the added phase - perhaps a left turn phase - on the delay at the intersection during the remaining part of the day.

Another situation which may also be considered is: if a change in traffic pattern occurs, what impact is caused by the continuation of the left turn phase operation, or is the left turn phase needed anymore? In his evaluation the traffic engineer should also assess the possible decrease in safety associated with the change. Also, the problem of driver expectancy arises. A motorist who has driven the same route a number of years expects a left turn phase. A sudden change in the signal operation may not be accompanied by a similar response on the drivers' part. Legal ramifications, in view of the rising number of suits against traffic departments, would need to be further researched.

An inherent problem in the addition of a left turn phase (or a deletion) is that most pre-timed signal controllers must keep the same number of phases and sequence thereof throughout the entire day. Adding, say, a left turn phase for only one period of the day is generally not possible. Therefore, a proposed addition of a phase should be evaluated as to its effects on delay for its entire period of operation.

Analysis Description

The intersection of Stadium Avenue and Northwestern Avenue was chosen for the analysis procedure. See Figure 17. A left turn phase currently operates at this intersection for northbound Northwestern Avenue traffic. There is no opposing left turn phase (for southbound traffic) and therefore the through northbound traffic is released along with the green left turn indication. In effect this phase arrangement represents a leading green for the northbound traffic. Currently, a separate left turn lane is designated by means of a channelizing island. This lane has a storage capacity of about five vehicles.

Using output statistics previously generated as well as simulation runs with revised timings, a statistical analysis was undertaken. The present signal timing with the left turn phase was simply transformed into an even distribution of green time for the two Northwestern Avenue approaches. The leading time allotted for northbound left and through traffic was added to the green time on the opposing approach; the left turn phase was thus deleted.

Four different scenarios for each period of the day - AM, Off, and PM peaks - were simulated. Each simulation was conducted on the arterial as a whole with individual link statistics used to analyze this particular intersection. Three simulation runs were used; each subinterval represented 10 minutes of real-time. The four scenarios were simulated as follows:

Existing timing with left turn phase;

Existing timing without left turn phase;

Proposed timing with left turn phase;

Proposed timing without left turn phase;

Upon consideration of the offsets along the arterial affected by the deletion of the phase, no changes were made for the present signal timing. Using proposed timings (Webster cycles and splits with delay/difference offsets) the delay/difference method was used to refine the offsets along the affected link.

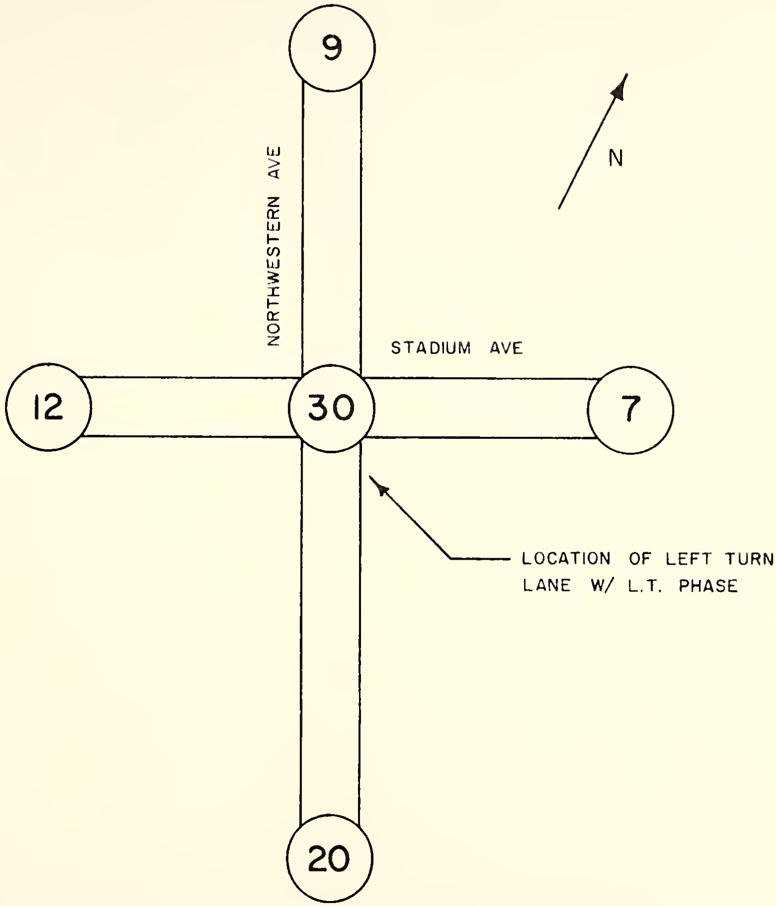

FIGURE 17: LINK AND NODE REPRESENTATION OF STUDY INTERSECTION

Statistical Comparison of the Delay

Within the existing timing plan with and without the left turn phase insignificant changes in the network-wide measures of effectiveness occurred. This was also the case within the proposed timing plans. Consideration of the individual study intersection resulted in interesting findings. Table 24 lists the average delay per vehicle in seconds experienced by the vehicles entering node 30 on all the approaches. Since it was expected that average delay per vehicle for the intersection as a whole would decrease, the hypotheses noted in Table 25 were chosen accordingly. Sample calculations are also shown in Table 25. As shown in Table 24, both the AM peak and off peak periods experienced significantly less total intersection delay when operated without the left turn phase. When the delay associated with the PM peak was tested at an $\alpha = 0.05$, no significant difference was detected in the delay with and without left turn phases. Consequently, it can be questioned whether or not the left turning traffic at this intersection is subject to an intolerable delay due to elimination of its left turn phase.

The delay of the link with the left turn phase (20, 30) was further analyzed both with the phase and without the phase. Table 26 shows the average delay per vehicle both before and after the change for the two timing plans. A two-sided t-test was conducted for each pair of delay values for the three periods of the day. Since the change in delay was unknown before conducting the tests the null hypothesis of $\mu_1 = \mu_2$ and the alternative hypothesis $\mu_1 \neq \mu_2$ were tested. For each of the six tests conducted there was no significant difference in delay for approach link (20,30). A level of significance of 0.05 was used.

By determining the amount of traffic traveling through the intersection during each of the peak periods, some indication of the travel time savings can be resolved. Approximately 11,500 vehicles traverse this intersection daily: 1530 during the morning peak, 8050 during the off peak hours, and 1920 during the evening peak hour. Further calculations show that if the left turn phase is removed in the existing timing scheme, 85 minutes of total travel time delay will be

Table 24. Delay Per Vehicle at Node 30 for All Approaches

	With Left Turn Phase (μ_1)	Without Left Turn Phase (μ_2)
Existing Timing		
AM**	16.86 $\pm$ 1.29	13.51 $\pm$ 1.15
Off**	10.48 $\pm$ 1.02	8.62 $\pm$ 0.81
PM	20.17 $\pm$ 1.96	19.06 $\pm$ 1.18
Proposed Timing		
AM**	14.67 $\pm$ 1.49	10.82 $\pm$ 1.08
Off**	8.34 $\pm$ 0.32	6.70 $\pm$ 0.57
PM	10.57 $\pm$ 1.11	9.98 $\pm$ 0.60

Note: Delay is measured in seconds/vehicle ($\pm$ standard deviation) averaged over 3 intervals of 10 minutes each.

**Rejected null hypothesis $\mu_1 = \mu_2$ in favor of H_1: $\mu_1 > \mu_2$ for $\alpha = 0.05$.

Table 25. Sample Test Calculation for Differences in Delay

$H_0: \mu_1 = \mu_2$ $H_1: \mu_1 > \mu_2$ $n_1 = n_2 = 3$ $\alpha = 0.05$

$$T = \frac{\overline{X}_1 - \overline{X}_2}{S_p\sqrt{\frac{1}{n_1} + \frac{1}{n_2}}}$$

$$S_p^2 = \frac{(n_1-1)S_1^{\,2} + (n_2-1)S_2^{\,2}}{n_1+n_2 - 2}$$

$$S_p^{\,2} = \frac{2(1.29)^2 + 2(1.15)^2}{4} = 1.493$$

$$T = \frac{16.86 - 13.51}{1.222\sqrt{\frac{1}{3} + \frac{1}{3}}} = 3.36$$

$$t_{\alpha,4} = 2.132$$

Decision: Since $T > t_\alpha$, reject H_0 in favor of H_1.

Table 26. Delay Per Vehicle at Node 30 on Approach Link (20, 30)

	With Left Turn Phase (μ_1)*	Without Left Turn Phase (μ_2)*
Existing Timing		
AM	12.67 $\pm$ 1.30	13.56 $\pm$ 1.44
Off	5.79 $\pm$ 0.69	6.90 $\pm$ 1.36
PM	25.56 $\pm$ 2.74	27.64 $\pm$ 1.53
Proposed Timing		
AM	14.35 $\pm$ 2.68	11.95 $\pm$ 2.63
Off	6.63 $\pm$ 0.07	6.28 $\pm$ 0.48
PM	8.38 $\pm$ 0.65	8.45 $\pm$ 1.16

Note: Delay is measured in seconds/vehicle ($\pm$ standard deviation) averaged over 3 intervals of 10 minutes each.

*Failed to reject null hypothesis H_0: $\mu_1 = \mu_2$ as opposed to the alternative H_1: $\mu_1 \neq \mu_2$ for $\alpha = 0.05$ in all cases.

saved per day in the morning peak and 250 minutes (4.2 hours) would be saved in travel time for the off peak hours daily. If consideration is given to the inclusion of a left turn phase under the proposed timing plans, a daily 100 minute savings in travel time would be realized during the morning peak when operation is without a left turn phase. Likewise, a 220 minute (3.7 hours) travel time savings per day would be possible for the off peak period.

A one-step betterment of the signal timing at the Northwestern-Stadium Avenue intersection would provide a sizeable reduction in the delay experienced in crossing the intersection. Comparing the existing timing (with left turn phase) and the proposed timing (without the left turn phase) the daily reduction in delay would amount to 16.4 hours: 2.5, 8.5, and 5.4 hours saved for the AM, Off, and PM peaks, respectively.

The foregoing discussion and analysis would certainly question the necessity of a left turn phase at the intersection considered. Before a decision was made as to whether or not the phase should be eliminated, one may want to consider past and current accident patterns and other aspects of safety associated with the proposed action. Conflicts for northbound left turn movements from Northwestern Avenue onto Stadium Avenue would probably increase. A further study looking into the number of stops per vehicle before and after and also the cost of implementation would be of importance. Another factor worthy of note is the possibility of redistributing the green time among the approaches to better handle the demand and therefore provide a decrease in total delay and an increase in the service level.

Analyses of this type before action is taken would lead to a more acceptable nature relative to traffic flow in urban areas. Of significant importance in analyzing impacts before implementation is the fact that changes to the system are highly unlikely once a plan has been put into use; additional time and money must then be invested for the corrective action. Admission of an error as well as the extra financial resources necessary to rectify a predicament would oftentimes be outweighed by the continuation of the system's operation in its less than optimal state.

CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

This project demonstrated the application of the traffic simulation model UTCS-1 in signal timing development. Emphasis was placed on signal systems on arterial roadways where fixed-time interconnected signals were the common controller of traffic. UTCS-1, which is rapidly becoming an accepted simulation model, was used as an analysis tool in the evaluation of specific signal control strategies which included energy, air pollution and delay considerations. A more direct approach and awareness of signal timing problems can be achieved by employing computer simulation to analyze generated alternatives before implementation. Surely, much time and energy is wasted at signals whose timing is inconsistent with prevailing traffic conditions as was pointed out in this study. The major objectives of this investigation were to demonstrate the use of the UTCS-1 model and to apply it in determining efficient signal settings for a specific study area.

Several findings and accomplishments are evident at the conclusion of this research relative to the study's scope of interest. They are summarized as follows:

1. The UTCS-1S model as modified at Purdue University appears to be a valid model that accurately replicates the flow of traffic along a signalized arterial given the necessary description of the traffic stream and roadway. Validations performed in this study served to further substantiate the model's reliability which was also verified by several past studies.
2. A detailed manual was developed and, as presented in Appendix A, should aid the prospective user in the data encoding process for the UTCS-1 input. A sample simulation run is provided for detailing the output capabilities of a simulation run.

3. A regression equation was used to estimate fuel consumption rate using network average speed as the independent variable. A system's energy efficiency characteristics may therefore be evaluated.
4. Utilizing an EPA Modal Analysis Emissions Model, regression equations using fuel consumption to predict the emissions of carbon monoxide were developed. An Indiana mix of automobile model years was used in developing an equation for predicting emissions for a typical 100 car sample given average fuel consumption in miles per gallon.
5. Network-wide measures of effectiveness were shown to have varied significantly when subject to different common cycle lengths. Generally and regardless of the offset plan used, as cycle length decreased MOE's improved, for the traffic conditions prevailing in the study area. Special attention must be given to critical elements such as pedestrian crossing time, cycle failures, and stops per vehicle. Safety should be a major consideration in selecting a cycle length.
6. The delay/difference of offset technique consistently produced higher network average speeds and less delay over a range of cycle lengths when compared with an unimproved signalized arterial.
7. A systematic approach is necessary for the proper timing of traffic signals whether dealing with an isolated intersection or with an arterial with several signalized intersections.
8. Use of simulation to assess the performance of various signal timing methodologies proved extremely useful and expedient. A try-and-review approach, while easy with computer simulation, can be hazardous and costly in the field.
9. Signal systems developed herein showed that, if implemented, would result in substantial savings of fuel and travel time while significantly reducing the emission of carbon monoxide. Also, spillback occurring along Grant Street would be alleviated. According to the specific community policies as

seen by the local traffic engineers, the signal timings should be updated as listed in Appendix D. For verification of the current traffic volumes, counts should be taken and compared with those that were used in the development of the signal timings. Those volumes at variance should be used to refine the recommended signal timings. Counts should also be taken to ascertain the occurrence of peak periods throughout the day for signal operation accordingly. Additionally, whenever volumes do not meet the warrants for signal operation i.e., late night operation, and when safety and intolerable delay problems would not arise, the signals should be placed on flashing operation.

10. An anlaysis of the left turn phase at the intersection of Stadium and Northwestern Avenues was performed to evaluate the need for that phase. A significant reduction in delay was shown when the intersection was simulated for the operation of the signal without a left turn phase. However, problems with safety would have to be further evaluated.
11. A detailed study should be conducted before initiation of a specific signal system particularly if it includes added or deleted phases. A problem with driver expectancy must be handled. Once a particular signal timing system has been implemented, corrective action or updating often are neglected. Emphasis should be placed on proper timing initially based on current volumes and turning movements. Also, undue delay to motorists at signals especially during periods of low volumes causes disrespect for traffic controls and should therefore be avoided whenever possible.
12. Familiarity with the street system to be timed is necessary. Complaints about the signal operation from motorists should be seriously considered and included in the development of new timings.

13. Further application of the UTCS-1 model is encouraged and, using the development in this project, additional studies of known problem areas should be investigated. Application of the model's output can be relied upon in judging the relative merits of a group of arterial operational alternatives. Of particular interest would be the application of the model to actuated signals in studying improvement methodologies. Additionally, more detailed research can be conducted relative to the conflicts of pedestrians and traffic particularly at intersections with the Right-Turn-On-Red provision. Further research is also needed regarding the utility of installing special phases to satisfy demand with high peaking characteristics. Also, skip-phase operation regarding delay and particularly safety could be further evaluated.
14. A worthwhile addition to the UTCS model would be a routine that calculates an arterial's acceleration noise. Given that a higher acceleration noise generally denotes an increase in the probability of accidents, a measure of effectiveness relative to safety would be a worthwhile addition.

These findings and recommendations serve to improve the urban environment through better traffic flow. Improvements through proper traffic management with little or no capital cost can increase urban transportation efficiency to a great extent, and such improvements must be considered before expensive alternatives are employed.

LIST OF REFERENCES

LIST OF REFERENCES

1. Ferlis, Robert A., and Worrall, Richard D., "Experimental Applications of the UTCS-1 Network Simulation Model", Transportation Research Record 567, 1976, pp. 45-55.

2. Bauer, Christian S., "Some Energy Considerations in Traffic Signal Timing", Traffic Engineering, February 1975, p. 23.

3. Peat, Marwick, Mitchell, and Co., and KLD Associates, "Network Flow Simulation for Urban Traffic Control System - Phase II", Springfield, Virginia: National Technical Information Service, 1973 (PB 230 760-230764/AS).

4. Lieberman, E. B., Worrall, R. D., and Bruggeman, J. M., "Logical Design and Demonstration of UTCS-1 Network Simulation Model", Highway Research Record 409, 1972, p. 47.

5. Courage, K. G. and Parapar, S. M., "Delay and Fuel Consumption at Traffic Signals", Traffic Engineering, November 1975, pp. 23-27.

6. Patterson, R. M., "Traffic Flow and Air Quality", Traffic Engineering, November 1975, pp. 14-17.

7. Ibid., p. 14.

8. Courage, K. G., and Parapar, S. M., "Delay and Fuel Consumption at Traffic Signals, op.cit.

9. Honeywell Traffic Management Center, Fuel Consumption Study: Urban Traffic Control System (UTCS) Software Support Project, No. FHWA-RD-76-81, Final Report, February 1976.

10. Courage, K. G. and Parapar, S. M., "Delay and Fuel Consumption at Traffic Signals", op.cit.

11. Automobile Exhaust Emission Modal Analysis Model, U. S. Environmental Protection Agency, Office of Air and Water Programs, January, 1974, EPA-460/3-74-005, p. 1.

12. Ibid.

13. Noll, K. E., "Transportation and Air Pollution", Traffic Engineering, June 1972, pp. 12-17.

14. "Transportation System Management", Transportation Research Board Special Report 172, 1977, p. 1.

15. Noll, K. E., "Transportation and Air Pollution", op.cit., p. 16.

16. Hall, Dana L., "Energy, Air Pollution, Delay, and Safety Evaluation of Non-Signalized Control at Low Volume Intersections", Ph.D. Thesis, Purdue University, August 1977.

17. Lieberman, E. B., Worrall, R. D., and Bruggeman, J. M., "Logical Design and Demonstration of UTCS-1 Network Simulation Model", op.cit., p. 18.

18. Ibid.

19. Peat, Marwick, Mitchell and Co., and KLD Associates, "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 5: Applications Manual for UTCS-1 Network Simulation Model, March 1974, p. 24.

20. Ibid., p. 26.

21. Ibid., p. 27.

22. Ibid., p. 28.

23. Peat, Marwick, Mitchell and Co., and KLD Associates, "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 4: User's Manual for UTCS-1 Network Simulation Model, March 1974, p. 5.

24. Hall, Dana L., "Energy, Air Pollution, Delay, and Safety Evaluation of Non-Signalized Control at Low Volume Intersections", op.cit., p. 15.

25. "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 5, op.cit., p. 8.

26. "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 4, op.cit., p. 8.

27. Ibid.

28. Drew, Donald R., Traffic Flow Theory and Control, New York: McGraw Hill Book Co., 1968, p. 281.

29. Peat, Marwick, Mitchell and Co., and KLD Associates, "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 1: Technical Report, March 1974, p. 171.

30. Ferlis, Robert A., and Worrall, Richard D., "Experimental Applications of the UTCS-1 Network Simulation Model", op.cit., p. 51.

31. Cohen, S. L., "Application of Network Simulation Models to the Analysis of Urban Intersection Performance", Report No. FHWA-RD-74-25, Final Report, September 1973, p. 37.

32. "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 4, op.cit., p. 110.

33. "Automobile Exhaust Emission Modal Analysis Model", op.cit.

34. Hall, Dana L., "Energy, Air Pollution, Delay, and Safety Evaluation of Non-Signalized Control at Low Volume Intersections", op.cit., pp. 17-37.

35. Ibid., p. 23.

36. "Automobile Exhaust Emission Modal Analysis Model", op.cit., p. 10.

37. "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 1, op.cit., p. 1.

38. Ferlis, Robert A., and Worrall, Richard D., "Experimental Applications of the UTCS-1 Network Simulation Model", op.cit., p. 49.

39. Ibid.

40. "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 1, op.cit., p. 217.

41. Ibid., p. 224.

42. Ferlis, Robert A., and Worrall, Richard D., "Experimental Applications of the UTCS-1 Network Simulation Model", op.cit., p. 51.

43. Kennedy, W. J., Farr, R. M., and Labrum, W. D., "Implementing Computer Simulation of Traffic Signal Systems", Traffic Engineering, July 1975, p. 48.

44. Box, Paul C., and Oppenlander, Joseph C., "Manual of Traffic Engineering Studies, Fourth Edition, Arlington, Va.: Institute of Traffic Engineers, 1976, pp. 106-112.

45. Automobile Exhaust Emission Modal Analysis Model, op.cit., p. 4.

46. Ibid., p. 5.

47. Ibid., p. 12.

48. Hall, Dana L., "Energy, Air Pollution, Delay, and Safety Evaluation of Non-Signalized Control at Low Volume Intersections", op.cit., p. 21.

49. Ibid., p. 20.

50. Ibid., p. 57.

51. Honeywell Traffic Management Center, Fuel Consumption Study: Urban Traffic Control System (UTCS) Software Support Project, op.cit.

52. Ibid., p. 5.

53. Hall, Dana L., "Energy, Air Pollution, Delay, and Safety Evaluation of Non-Signalized Control at Low Volume Intersections", op.cit., p. 33.

54. Wagner, F. A., Barnes, F. C., and Gerlough, D. L., "Improved Criteria for Traffic Signal Systems in Urban Networks", NCHRP Report 124, 1971, p. 3.

55. Webster, F. V., "Traffic Signal Settings", London: Her Majesty's Stationery Office, Road Research Technical Paper No. 39, 1958.

56. Kreer, John B., "When Mixed Cycle Length Signal Timing Reduces Delay", Traffic Engineering, March 1977, pp. 17-20.

57. Wagner, F. A., Gerlough, D. L., Barnes, F. C., "Improved Criteria for Traffic Signal Systems on Urban Arterials", NCHRP Report 73, 1969, p. 6.

58. Ibid.

59. Ibid., p. 28.

60. Little, J. D. C., Martin, B. V., and Morgan J. T., "Synchronizing Signals for Maximal Bandwidth", Highway Research Record 118, 1966, pp. 21-45.

61. Ibid.

62. "The Subject of Pretimed Intersection Control", engineering information published by Eagle Signal Corporation, June 1976.

63. Lieberman, E. B., Worrall, R. D., and Bruggeman, J. M., "Logical Design and Demonstration of UTCS-1 Network Simulation Model", op.cit., p. 49.

64. "Network Flow Simulation for Urban Traffic Control System - Phase II", Volume 4, op.cit., pp. 124-138.

65. Ibid., p. 54.

66. Sinha, K. C., "Air Pollution Considerations in Traffic Planning and Engineering", Proceedings of the 61st Annual Road School, Purdue University, Bulletin No. 146 (Extension Series) 1975, p. 166.

General References

Anderson, Virgil L. and McLean, Robert A., Design of Experiments: A Realistic Approach, New York: Marcel Dekker, Inc., 1974.

Baerwald, John E., ed., Transportation and Traffic Engineering Handbook, Englewood Cliffs, New Jersey: Prentice-Hall, Inc., 1976.

Davies, Gary W., "Optimization of a Traffic Signal System Through Computer Simulation", Joint Highway Research Project No. 8, Purdue University, June 1972.

Highway Capacity Manual, Highway Research Board Special Report 87, 1965.

Lee, Bumjung, Crowley, K., and Pignataro, L. J., "Better Use of Signals Under Oversaturated Flows", Transportation Research Board Special Report 153, 1975.

Lieberman, Edward B., and Cohen, S. L., "New Techniques for Evaluating Urban Traffic Energy Consumption and Emissions", Transportation Research Record 599, 1976.

Little, J. D. C., "The Synchronization of Traffic Signals by Mixed-Integer Linear Programming", Operations Research, Vol. 14, No. 4, 1966, pp. 568-594.

McShane, W. R. and Pignataro, Louis J., "Some Results on Guidelines for Treatment of Traffic Congestion on Street Networks", Transportation Research Record 597, 1976.

Neter, John and Wasserman, William, Applied Linear Statistical Models, Homewood, Illinois: Richard D. Iwrin, Inc., 1974.

Pontier, Walter E., Miller, Paul W., and Kraft, Walter H., "Optimizing Flow in Existing Street Networks", NCHRP Report 113, 1971.

Snedecor, George W. and Cochran, William G., Statistical Methods, Sixth Edition, Ames, Iowa: The Iowa State University Press, 1967.

"Traffic Control in Oversaturated Street Networks", NCHRP Project 3-18(2), Final Report, June 1975.

Walpole, Ronald E. and Myers, Raymond H., Probability and Statistics for Engineers and Scientists, New York: MacMillan Publishing Co., Inc., 1972.

APPENDICES

APPENDIX A

APPENDIX A

UTCS-1 DATA REQUIREMENTS AND INPUT FORMAT

The UTCS model has been revised several times in order to facilitate data inputs. Much of the data required for input to the model can be acquired from existing traffic engineering department records. Familiarity with the street network to be simulated is advisable to assist in input data error detection. One would easily detect unusual turning movements or queue delay if actual flow characteristics were known. For this reason, it is suggested that the present system should be modeled and analyzed before new alternatives are simulated. The simulated measures of effectiveness, i.e. average speed, cycle failures, must closely correspond with field conditions before proceeding to test new signal timing schemes.

The urban street network to be simulated is separated into a network of one-way links (streets) and nodes (intersections) as depicted in Figure A-1. The current UTCS model has a capacity of 60 nodes and 85 links. Each link can accommodate up to 5 moving lanes. A two-way street is input as two one-way streets. For example, Main Street between two intersections (nodes 1 and 2) would be input using two links (1,2) and (2,1). Respective characteristics of each link would then also be included as data input.

Data Requirements

To adequately simulate an urban street network, the following input data are required (63):

1. For each link in the network: number of moving lanes, link length, turning movements, target free flow speed, mean queue discharge rate, downstream nodes from each link, pedestrian volume ranges, lane channelization, capacity of turn pockets;

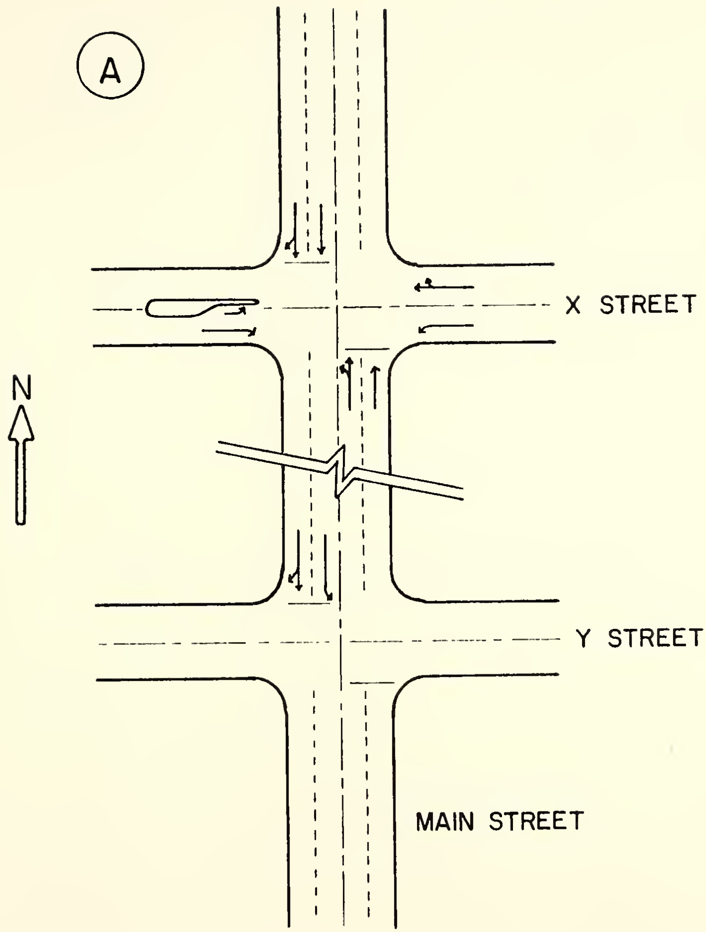

FIGURE A-1 STREET NETWORK

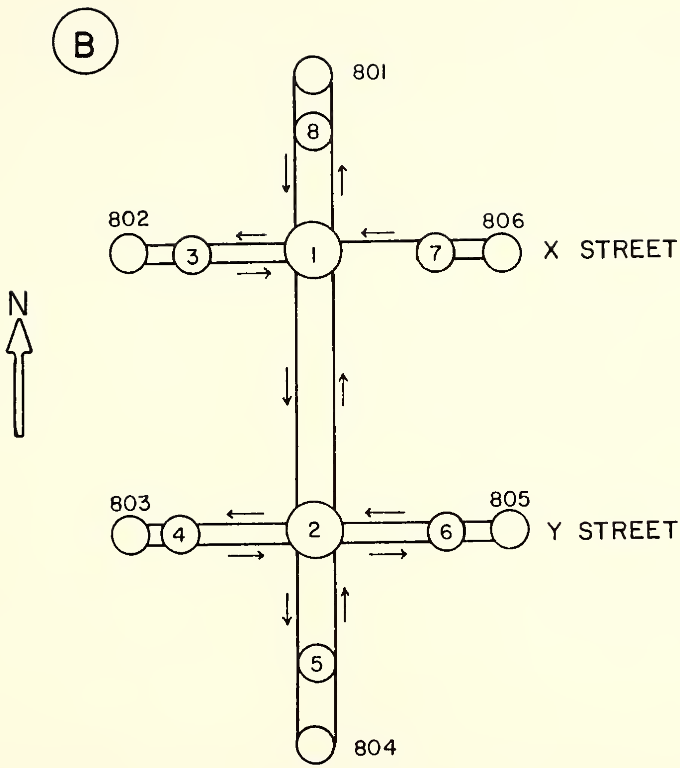

FIGURE A-1 STREET NETWORK

2. At each node (intersection): sequence and duration of signal or sign control for each appraoch, signal offsets along arterials;
3. Volume of traffic emitted onto network in vehicles per hour, percentage of trucks;
4. Duration of interval of time to be simulated.

The UTCS model utilizes several distributions to determine, among others, amber phase response, acceptable gaps at stop signs, and acceptance gaps for left-turning vehicles. These embedded calibration data may be revised by the user to more accurately simulate the driving behavior of specific area drivers. Overall, however, the UTCS model adequately simulates traffic on urban street networks with the existing embedded data.

As part of the output of the UTCS model, the input data is succinctly listed for easy verification of network coding. Figures A2 through A4 show sample format of the ordered input. Also printed before any execution of the simulation begins is the actual data deck input by the user. See Figure A5. UTCS performs diagnostic checking for proper input format and consistency of the coded data. For example, an approach to an intersection may not execute left turns if this is contrary to a previously established left turn prohibition. Appropriate error messages are printed if inconsistencies occur. These self explanatory messages facilitate debugging of the input data.

Model Output

The printed output gives cumulative statistics for the simulated interval. Data is provided for each network link then aggregated for the entire network. Figure A6 shows the link statistics as well as the network statistics. Measures of effectiveness for the simulated interval included in the link statistics are as follows (64):

- LINK — Link identification, by origin and destination node.
- VEH-MILES — Estimate of total vehicle miles of travel.
- VEH TRP — Count of vehicles discharged onto link.
- MOV TIME V-MIN — Total vehicle moving time in vehicle minutes.

SIMULATION OF TRAFFIC

THE UTCS-1 MODEL

UTCS-1S SAMPLE SIMULATION RUN

SAMPLE STREET , ANYTOWN, STATE , USA -0 08/ /77

SEED FOR RANDOM NUMBER GENERATOR IS 7581

LINK	LANE	SPAN	POCK L	POCK R	MEAN U-F	H	TURNING MOVEMENTS LEFT	TURNING MOVEMENTS THRU	TURNING MOVEMENTS RT	TURNING MOVEMENTS DIAG	DESTINATION NODES LEFT	DESTINATION NODES THRU	DESTINATION NODES RT	DESTINATION NODES DIAG	LOST	PED DEN	LANE CHAN 1	LANE CHAN 2	LANE CHAN 3	LANE CHAN 4	LANE CHAN 5	TYPE	G	L
(801, 8)	2	200	0	0	ENTRY	21	0	100	0	0	0	1	0	0	37		0	0	0	0	0	1	2	1
(1, 2)	2	650	0	0	30	21	25	60	15	0	6	5	4	0	37	0	0	1	0	0	0	1	2	2
(2, 5)	1	360	0	0	30	21	0	100	0	0	0	804	0	0	37	0	0	0	0	0	0	1	2	3
(8, 1)	2	300	0	0	30	21	0	85	15	0	0	2	3	0	37	0	0	0	0	0	0	1	2	4
(803, 4)	1	200	0	0	ENTRY	21	0	100	0	0	0	2	0	0	37		0	0	0	0	0	1	2	5
(805, 6)	1	200	0	0	ENTRY	21	0	100	0	0	0	2	0	0	37		0	0	0	0	0	1	2	6
(1, 8)	2	360	0	0	30	21	0	100	0	0	0	801	0	0	37	0	0	0	0	0	0	1	2	7
(4, 2)	1	300	0	0	30	21	10	81	9	0	1	6	5	0	37	0	0	0	0	0	0	1	2	8
(6, 2)	1	300	0	0	30	21	10	60	30	0	5	4	1	0	37	0	0	0	0	0	0	1	2	9
(802, 3)	1	200	0	0	ENTRY	21	0	100	0	0	0	1	0	0	37		0	0	0	0	0	1	2	10
(2, 4)	1	380	0	0	30	21	0	100	0	0	0	803	0	0	37	0	0	0	0	0	0	1	2	11
(2, 6)	1	360	0	0	30	21	0	100	0	0	0	805	0	0	37	0	0	0	0	0	0	1	2	12
(3, 1)	1	300	3	0	30	21	35	0	65	0	8	0	2	0	37	0	0	0	0	0	0	1	2	13
(804, 5)	1	200	0	0	ENTRY	21	0	100	0	0	0	2	0	0	37		0	0	0	0	0	1	2	14
(806, 7)	2	200	0	0	ENTRY	21	0	100	0	0	0	1	0	0	37		0	0	0	0	0	1	2	15
(1, 3)	1	380	0	0	30	21	0	100	0	0	0	802	0	0	37	0	0	0	0	0	0	1	2	16
(5, 2)	1	300	0	0	30	21	8	85	7	0	4	1	6	0	37	0	0	0	0	0	0	1	2	17
(7, 1)	2	300	0	0	30	21	30	60	10	0	2	3	8	0	37	0	0	1	0	0	0	1	2	18
(2, 1)	2	610	0	0	30	21	11	89	0	0	3	8	0	0	37	0	0	0	0	0	0	1	2	19

Figure A2. Program Listing of Input Link Data

```
                                      TRAFFIC SIGNAL DATA

NODE   INTVL   DURATION     OFFSET                    SIGNAL CODES FACING INDICATED APPROACH
                                            (  8,  1)    (  3,  1)    (  2,  1)    (  7,  1
  1      1     32  ( 53P)    0  (  0P)          1            5            1            5
  1      2      3  (  5P)   32  ( 53P)          0            5            0            5
  1      3     22  ( 37P)   35  ( 58P)          5            1            2            1
  1      4      3  (  5P)   57  ( 95P)          5            0            2            0

NODE   INTVL   DURATION     OFFSET                    SIGNAL CODES FACING INDICATED APPROACH
                                            (  1,  2)    (  4,  2)    (  5,  2)    (  6,  2
  2      1     32  ( 53P)   15  ( 25P)          1            5            1            5
  2      2      3  (  5P)   47  ( 78P)          0            5            0            5
  2      3     22  ( 37P)   50  ( 83P)          5            1            5            1
  2      4      3  (  5P)   12  ( 20P)          5            0            5            0
                                   NODE  3 IS UNDER SIGN CONTROL

NODE   INTVL   DURATION     OFFSET                    SIGNAL CODES FACING INDICATED APPROACH
                                            (802,  3)    (  1,  3)    (
  3      1     80  (100P)    0  (  0P)          1            1
                                   NODE  4 IS UNDER SIGN CONTROL

NODE   INTVL   DURATION     OFFSET                    SIGNAL CODES FACING INDICATED APPROACH
                                            (803,  4)    (  2,  4)    (
  4      1     80  (100P)    0  (  0P)          1            1
                                   NODE  5 IS UNDER SIGN CONTROL

NODE   INTVL   DURATION     OFFSET                    SIGNAL CODES FACING INDICATED APPROACH
                                            (804,  5)    (  2,  5)    (
  5      1     80  (100P)    0  (  0P)          1            1
                                   NODE  6 IS UNDER SIGN CONTROL
```

Figure A3. Program Listing of Signal Input Data

SUB-INTERVAL 1

ENTRY LINK STATISTICS

LINK	GRN FLOW RATE (VEH/HR)	RED FLOW RATE (VEH/HR)	PCT. TRUCKS
(801, 8)	450	-0	2
(803, 4)	250	-0	2
(805, 6)	275	-0	2
(806, 7)	450	-0	2
(802, 3)	250	-0	2
(804, 5)	425	-0	2

Figure A4. Program Listing of Volume Input Data

CARD FILE LIST

```
UTCS-1S SAMPLE SIMULATION RUN                                                          0
SAMPLE STREET        ANYTOWN,STATE           USA                   08    77            1
-550 0900                                                                              3
801  8 20000 0  0  1        1  2 65000 0  6  5  4     2  5 36000 0   804               4
  8  1 30000 00 0  2  3   803  4 20000 0     2      805  6 20000 0     2               4
  1  8 36000 0   801         4  2 30000 0  1  6  5     6  2 30000 0  5  4  1           4
802  3 20000 0     1         2  4 38000 0   803        2  6 36000 0   805              4
  3  1 30000 3  8  0  2   804  5 20000 0     2      806  7 20000 0     1               4
  1  3 38000 0   802         5  2 30000 0  4  1  6     7  1 30000 0  2  3  8           4
  2  1 61000 0  3  8                                                                   4
801  8 2   2137            1  2 2 302137      1      2  5 1 302137                     5
  8  1 2 302137          803  4 1   2137          805  6 1   2137                      5
  1  8 2 302137            4  2 1 302137             6  2 1 302137                     5
802  3 1   2137            2  4 1 302137             2  6 1 302137                     5
  3  1 1 302137          804  5 1   2137          806  7 2   2137                      5
  1  3 1 302137            5  2 1 302137             7  1 2 302137     1               5
  2  1 2 302137                                                                        5
801  8   0100   0       1  3  0100   0        2  4  0100   0       6  2 10 60 30       7
  8  1   0 85 15        2  1 11 89   0      804  5  0100   0       2  6  0100   0      7
  1  8   0100   0       1  2 25 60 15         5  2  8 85   7     806  7  0100   0      7
802  3   0100   0     803  4  0100   0        2  5  0100   0       7  1 30 60 10       7
  3  1 35   0 65        4  2 10 81   9      805  6  0100   0                           7
  3  1  7    7  1  3    8  1  2    4  2  6   1  2  5    5  2  1    6  2  4             8
  1      8  3  2  7        321515    30505   225121    35020                          10
  2 15   1  4  5  6        321515    30505   225151    35050                          10
  3    802  1                  11                                                     10
  4    803  2                  11                                                     10
  5    804  2                  11                                                     10
  6    805  2                  11                                                     10
  7    806                      1                                                     10
  8    801  1                  11                                                     10
801  8  450       2 803  4  250      2 805  6  275      2 806  7  450       2         20
802  3  250       2 804  5  425      2                                                20
1000                            1                                                     60
```

MAXIMUM INITIALIZATION PERIOD =-550 SECONDS.

Figure A5. Listing of Data Deck

CUMULATIVE STATISTICS SINCE BEGINNING OF SIMULATION

PRESENT TIME IS 9 16 40, ELAPSED SIMULATED TIME IS 16 MINUTES, 40 SECONDS

LINK STATS FOR VEHICLES DISCHARGED

LINK	VEH-MILES	VEH TRP	MOV. TIME V-MIN	TOTAL DELAY V-MIN	M/T	TOTAL TIME V-MIN	T-TIME / VEH. SEC	TOT-DLY / VEH. SEC	AVG. SPEED MPH	AVG. OCC.	STOPS / VEH	AVG SAT PCT	CYCL FAIL	STOP DELAY V-MIN	STP-DLY / VEH. SEC	QUEUE DELAY V-MIN	QUE-DLY / VEH. SEC
(1, 2)	23.19	190	47.0	39.6	.54	86.6	27.4	12.5	16.1	5.2	.71	9	0	20.4	6.5	28.5	9.0
(2, 5)	8.63	127	17.7	5.6	.76	23.2	11.0	2.6	22.3	1.4	.02	8	0	.1	.1	.6	.3
(8, 1)	7.27	128	14.5	23.5	.38	38.0	17.8	11.0	11.5	2.2	.48	7	0	11.8	5.5	14.1	6.6
(1, 8)	10.31	152	21.1	7.6	.73	28.7	11.3	3.0	21.6	1.7	.05	5	0	.4	.2	2.3	.9
(4, 2)	3.92	69	8.1	25.7	.24	33.9	29.4	22.4	6.9	2.0	.86	13	0	17.9	15.5	21.2	18.4
(6, 2)	4.20	74	8.9	26.2	.25	35.2	28.5	21.3	7.2	2.1	.77	14	1	17.9	14.6	21.1	17.1
(2, 4)	5.91	84	11.8	4.1	.74	15.8	11.3	2.9	22.4	.9	.06	6	0	.1	.1	.5	.4
(2, 6)	7.60	112	15.5	6.3	.71	21.8	11.7	3.4	20.9	1.3	.10	8	0	.3	.2	1.5	.6
(3, 1)	3.81	67	8.0	13.1	.38	21.1	18.9	11.8	10.8	1.3	.70	7	0	5.8	5.2	6.7	6.0
(1, 3)	7.40	104	15.0	5.3	.74	20.3	11.7	3.1	21.9	1.2	.05	7	0	.2	.1	1.2	.7
(5, 2)	6.65	117	13.6	27.2	.33	40.7	20.9	13.9	9.8	2.4	.60	16	0	15.5	7.9	19.9	10.2
(7, 1)	6.93	122	14.4	37.0	.28	51.5	25.3	18.2	8.1	3.0	.76	10	0	23.5	11.5	27.9	13.7
(2, 1)	15.15	132	30.1	38.3	.44	68.4	31.1	17.4	13.3	4.1	.76	7	0	25.5	11.6	31.2	14.2

NETWORK STATISTICS

Figure A6. Link and Network Statistics

```
VEHICLE-MILES= 110.97      VEHICLE-MINUTES=   485.2      VEHICLE-TRIPS (EST.)=  581      STOPS/VEHICLE= 1.13
MOVING/TOTAL TRIP TIME= .465   AVG. SPEED (MPH)=13.72   MEAN OCCUPANCY=  28.9 VEH.   AVG DELAY/VEHICLE= 26.80 SEC
TOTAL DELAY=  259.5 MIN.       DELAY/VEH-MILE= 2.34 MIN/V-MILE       TRAVEL TIME/VEH-MILE= 4.37 MIN/V-MILE
STOP DELAY=  139.5 MIN.   STOP DELAY/VEH= 14.41 SEC.   QUEUE DELAY=  176.8 MIN.   QUEUE DELAY/VEH= 18.25 SEC.
SEED FOR RANDOM NUMBER GENERATOR IS   86283703
```

Figure A6. Continued

- TOTAL DELAY V-MIN	Total delay time computed as the difference between total travel time and ideal travel time based on the target speed for link, in vehicle minutes.
- M/T	Ratio of moving time at desired speed to total time for link.
- TOTAL TIME V-MIN	Total travel time for link in vehicle minutes.
- T-TIME/VEH SEC	Average travel time per vehicle for link in seconds.
- TOT-DLY/VEH SEC	Average total delay per vehicle for link in seconds.
- AVG SPEED MPH	Average traffic speed for link (space mean speed) in miles per hour.
- AVG OCC	Average occupancy for link in vehicles.
- STOPS/VEH	Percentage of vehicles stopping at least once by link.
- AVG SAT PCT	Average saturation percentage for link expressed as the average over time of the length of the link which is occupied by vehicles divided by its total storage capacity.
- CYCL FAIL	Total number of cycle failures by link defined as the number of times a queue fails to clear from the discharge end of the link during a green period.
- STOP DELAY V-MIN	Delay due to stopping in vehicle minutes.
- STP-DLY/VEH SEC	Average stopped delay per vehicle in seconds.
- QUEUE DELAY V-MIN	Delay due to approaching and moving in a queue expressed in vehicle-minutes.
- QUE-DLY/VEH SEC	Average queue delay per vehicle in seconds.

By monitoring these various measures of effectiveness, the traffic engineer can generate alternatives and test their validity via simulation.

Input Format

As seen in Figure A5 the UTCS input data must follow a precise format. Data input requirements are grouped according to function as follows (65):

Identification cards:
- title cards,
- network name card,

Link cards:
- link geometry cards,
- link operation cards,
- link turning movement cards,
- opposed link identification cards,

Signal or Sign control cards;

Flow rate cards;

Control cards;

Embedded Data Change cards*

Each card is identified by its type ranging from Type 00 to Type 99. Note that all input data are integers - no decimal points may be punched. The following input data format describes specifically the use of UTCS-1S as applied to urban arterial fixed signal timing pattern analysis. Further information, input format for actuated signals, and changes to embedded data can be found in Vol. 4, pp. 56-97 of the UTCS Manual. Considerable attention must be given to data input preparation. The format for input data is as follows:

Execution Control Card - Type 99

This card must be the first card in the data set. It contains a collection of codes which identify the mode of execution chosen by the user. As modified, the UTCS program will execute using the following format:

---

*Optional

Cols.	Data Word
1	1
9	1
11,12	12
15	1
25	1
79,80	99 (Card Type)

Title Card - Type 00

Cols.	Data Word
1-65	Title identifying this run
70-77	Seed for random number generator (optional)
79,80	00

Any alphanumeric title identifying this study may be punched. The seed for the random number generator may be any odd integer (not a multiple of 5) of up to 8 digits in length. If left blank, the model will assign a seed of 7581.

Network Name Card - Type 01

Cols.	Data Word
1-20	Network Name (location, type, etc.)
21-40	Name of city
45-56	Name of state
63,64	Month
66,67	Day
69,70	Year
79,80	01

Network Priming Card - Type 03

Cols.	Data Word
1-4	Maximum initialization time, seconds
6-9	Clock time, in hours and minutes (24 hour clock time)
79,80	03 (Card Type)

For accurate simulation results, the network should be in equilibrium (veh entry $\simeq$ veh exit). The maximum initialization time discharges vehicles onto the network without collecting statistics. If equilibrium is not attained within this maximum initialization time, a warning message will be printed and the model will abort. If the user desires to commence simulation even if equilibrium has not been attained, the value of maximum initialization should be entered as a negative integer.

Link Geometry Cards - Type 04

These cards defined the geometry of all entry and internal links. (Vehicles are emitted onto network via entry links).

Cols.	Data Word
1-3	Upstream node number, i, of link (i,j)
4-6	Downstream node number, j, of link (i,j)
7-10	Length of network link, feet.
11	Grade code denoting the grade of the link as follows:

Grade Code	Grade G (%)
1	$G \leq -5$
2	$-5 < G \leq 0$
3	$0 < G \leq 1$
4	$1 < G \leq 2$
5	$2 < G \leq 3$
6	$3 < G \leq 4$
7	$4 < G \leq 5$
8	$5 < G$

(blank or 0 defaults to 0%)

Cols.	Data Word
12	Capacity of right turn pocket, vehicles (autos)
13-14	Capacity of left-turn pocket, vehicles (autos)

Cols.	Data Word
15-17	Downstream node, K, of link (j, K) which receives left-turning traffic from link (i,j)
18-20	Downstream node, ℓ, of link (j,ℓ) which receives through traffic
21-23	Downstream node, m, of link (j,m) which receives right turning traffic.
24-26	Downstream node, n, of diagonal link (j,n) which receives either: left half turning traffic: input is negative (i.e., -n) (node number cannot exceed 99) or: right half turning traffic: input is positive (i.e., n).
27-52	Identical to cols. 1-26.
53-78	Identical to cols. 1-26.
79,80	04 (Card Type).

Explanatory Notes:

There are three types of links:

- Internal links, all statistics accumulated pertain to these links; they are internal to the network.
- Entry links, serve to introduce vehicles into the network. Vehicles are emitted from entry nodes at specified flow rates when signals and traffic conditions permit. No statistics are accumulated for entry links. The upstream node number, i, of all entry links (i,j) must be numbered, $i \geq 800$.
- Exit links, receive all vehicles discharged from the network. They are not specified on the link geometry cards but as downstream nodes of either entry or internal links. The downstream node number, j, of all exit links (e,j) must be numbered, $j \geq 800$.

Cols. 1-3	The node number, i, is entered; $i \leq$ MXND* for internal links. For entry links $i \geq 800$.
Cols. 4-6	The node number, j, is entered; $j \leq$ MXND*.
Cols. 7-10	The length L_{ij} of link (i,j) in feet is entered. This distance extends from stop line to stop line. See Figure A7.
Cols. 12,13-14	Capacity of turn pockets is expressed in number of vehicles - (length of pocket)/20'.

Link Operation Cards - Type 05

Cols.	Data Word
1-3	Upstream node number, i, of link (i,j).
4-6	Downstream node number, j, of link (i,j).
8	Number of moving lanes (≤ 5).
10,11	Desired free flow speed, U_f, on internal links, mph. Leave blank for all entry links.
12,13	Queue discharge rate, mean headways, in tenths of a second, i.e., input XX implies X.X seconds per vehicle (auto).
14,15	Lost time or queue start-up delay, in tenths of a second.
17	Code denoting intensity of pedestrian volume crossing the street at the upstream end of internal link (i,j) i.e., at node i.

*MXND (maximum node number) is equal to 60 in this model.

Code	Pedestrian Volume
0 (blank)	Light (100-250 peds/hr)
1	Moderate (250-500 peds/hr)
2	Heavy (above 500 peds/hr)
3	No pedestrian traffic

Cols.	Data Word
19	Code denoting channelization of lane 1 (curb lane) on link (i,j):

Code	Channelization
0 (blank)	Unrestricted
1	Reserved for left-turn vehicles
4	Reserved for right-turn vehicles

Cols.	Data Word
20	Channelization code for lane 2 (if any)
21	Channelization code for lane 3 (if any)
22	Channelization code for lane 4 (if any)
23	Channelization code for lane 5 (if any)
26-50	Identical to Cols. 1-25 for next link.
51-75	Identical to Cols. 1-25 for next link.
79,80	05 (Card Type).

Explanatory notes:

Col. 8 The number of moving lanes on link (i,j); a parking lane (if any) is not included, nor is a left or right turn pocket. Maximum number of lanes is: 5 with no turn pockets; 4 with 1 pocket, and 3 with 2 pockets.

Cols. 12,13 The mean time gaps (headway) between vehicles discharging from a standing queue is entered. This value in tenths of a second, applies only to those vehicles who were fourth in the queue or further downstream. (Typical values 21-25).

Cols. 14-15 This value, expressed in tenths of a second is the delay experienced by all load vehicles in queues, when responding to a signal phase change from red to green. (Typical value 37).

Cols. 19-23 Restrictions on channelization coding:

- A link with a single lane cannot have any channelized lanes.
- A parking lane is not considered a moving lane.
- At least one lane in a link which can have a through movement must be unrestricted.
- Lane 1 is the outside lane nearest the right-hand side of the street.
- A left or right turn pocket is not specifically channelized and is not considered a lane.

Link Turning Movements - Type 07

These cards specify the turning movements for each entry and internal network link. These movements are specified as percentages over some period of time.

Cols.	Data Word
1-3	Upstream node number, i, of link (i,j).
4-6	Downstream node number, j, of link (i,j).
7-9	Percentage of vehicles turning left at node j upon exiting from link (i,j).
10-12	Percentage of vehicles travelling through at node j.
13-15	Percentage of vehicles turning right at node j.
16-18	Percentage of vehicles turning diagonally at node j.
21-38	Same as cols. 1-18 for next link.
41-58	Same as cols. 1-18 for next link.
61-78	Same as cols. 1-18 for next link.
79,80	07 (Card Type)

Auxiliary Topology Card - Type 08

These cards verify the links that have opposing left turning traffic that would serve as impedance to the free flow of through traffic. Only links with opposing left turners need to be specified.

Cols.	Data Word
1-3	Upstream node number, i, of link (i,j) which carries left turning traffic as specified on its Type 07 Card.
4-6	Downstream node number, j, of link (i,j).
7-9	Upstream node number, p. of opposing link (p,j) whose through traffic impedes left turners discharging from (i,j).
11-19	Same as cols. 1-10 for other links.
21-29	Same as cols. 1-10.
.	
.	
.	
.	
61-69	Same as cols. 1-10.
79-80	08 (Card Type).

Signal Cards: Fixed Time Control*- Type 10

The signal control at each node (intersection) is completely specified on one signal card. Signal or sign control must be specified for every node in the network. Uncontrolled intersections are specified by yield signs facing minor approaches and perpetual green facing the major approaches.

Cols.	Data Word
1-3	Node number identifying intersection of network. This node is also the

*For actuated signals and signals with more than six intervals see Vol. 4, pp. 70-82 , Reference 23.

	downstream node of all links entering this intersection.
5-7	Reference offset to Interval 1, seconds.
8-10	Upstream node number of approach link number 1.
11-13	Upstream node number of approach link number 2.
14-16	Upstream node number of approach link number 3.
17-19	Upstream node number of approach link number 4.
20-22	Upstream node number of approach link number 5.
26-28	Duration of Interval 1, seconds. STOP or YIELD sign: Leave blank.
29	Control code for signal facing approach link number 1 during Interval 1.
30	Control code for signal facing approach link number 2 during Interval 1.
31	Control code for signal facing approach link number 3 during Interval 1.
32	Control code for signal facing approach link number 4 during Interval 1.
33	Control code for signal facing approach link number 5 during Interval 1.
35-42	Same data as in cols. 26-33, but for Interval 2 (only one interval is input for sign control)
44-51	Same data as in cols. 26-33, but for Interval 3.

Cols.	Data Word
53-60	Same data as in cols. 26-33, but for Interval 4.
62-69	Same data as in cols. 26-33, but for Interval 5.
71-78	Same data as in cols. 26-33, but for Interval 6.
79-80	10 (Card Type).

Explanatory Notes:

While the approaches to an intersection may be coded in arbitrary sequence, Figure A7 serves to facilitate proper coding.

Cols. 1-3	In Figure A8 this would be "n".
Cols. 5-7	This value provides the means for coding progressive signal operation as well as other methods of synchronization of signals in a network.
Cols. 8-10	Depicted as "i".
Cols. 11-13	Depicted as "j".
Cols. 14-16	Depicted as "k".
Cols. 17-19	Depicted as "ℓ".
Cols. 20-22	This is blank in this case since there are only four approaches.

Code	Control
0	Yield or Amber.
1	Green
2	Red
3	Red with green right arrow.
4	Red with green left arrow.
5	Stop or red with right turn permitted.
6	Red with green diagonal arrow.
7	No turns-green thru arrow.
8	Red with left and right green arrows.
9	No left turn-green thru and right.

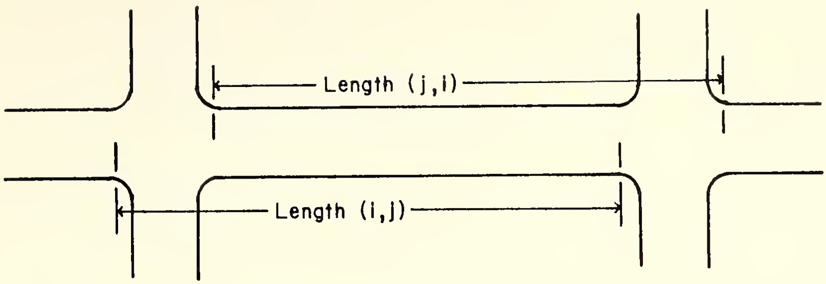

FIGURE A-7 LINK LENGTH DETERMINATION

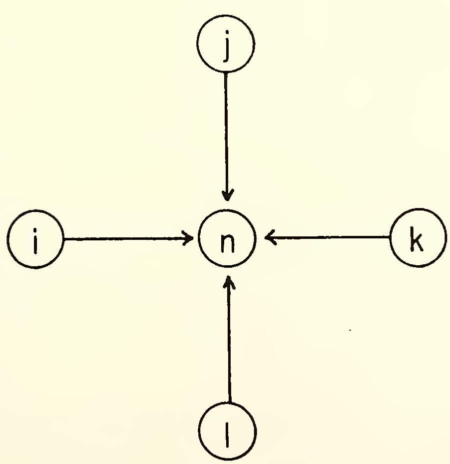

FIGURE A-8 SIGNAL CODING SCHEMATIC

If the intersection is signed (STOP or YIELD), and not signalized, then the following rules apply:

1. Set cols. 5-7, 26-28, and 35-78 blank.
2. Set proper code (either 5 or 0) for the signed links. Code = 1 for unsigned main links.
3. All other input data as indicated.

<u>Flow Rate Cards - Type 20</u>

Provisions have been included in UTCS for consideration of the platooning effects caused by signals at upstream intersections. If it is desired to utilize this option, data is needed on the volume which entered the link connecting the intersection to be simulated and the nearest upstream signalized intersection during the upstream green and yellow phase, denoted by V_g (primary flow). V_g and V_r are defined in terms of vehicles per hour of green. See Figure A9.

Cols.	Data Word
1-3	Upstream node of the entry link (i,j).
4-6	Downstream node of the entry link (i,j). (This node must be the aforementioned nearest signalized intersection).
8-11	V_g in veh/hr of green.
13-16	V_r in veh/hr of green.
17-18	Percent trucks.
20-37	Same as 1-18 for next entry link.
39-56	Same as 1-18 for next entry link.
58-75	Same as 1-18 for next entry link.
79-80	20 (Card Type).

If the platooning option is not to be utilized, set $V_r = 0$, V_g = flow volume.

<u>Simulation Control Card - Type 60</u>

This card is always the final card for each simulation interval.

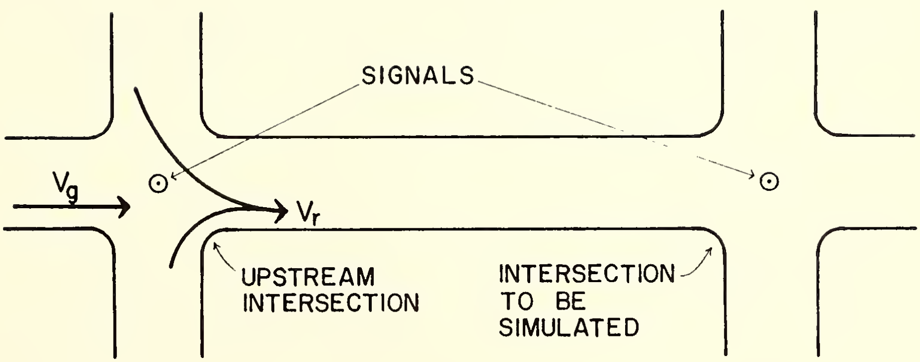

FIGURE A-9 PRIMARY (V_g) AND SECONDARY (V_r) FLOW

Cols.	Data Word
1-4	Duration of simulation interval.
79-80	60 (Card Type).

Selecting Alternatives

To this point only the UTCS model, which does not output any fuel consumption or emission characteristics, has been used. Using an automobile exhaust emission modal analysis model in conjunction with the UTCS model, regression equations useful to predice exhaust emissions and fuel consumption were developed. Together with the output of UTCS and these equations (see page 47) one can generate signal timing alternatives with delay, emissions, and fuel consumption characteristics as measures of effectiveness. Indeed, the reduction of one may not necessarily minimize another. One must set goals or tolerance levels and strive for the 'best choice' alternative.

Sample Simulation Run

The small network in Figure A1 will be used here to demonstrate the UTCS simulation model. Figure A1A is the actual roadway to be simulated. Data cards have been punched and input to the UTCS model. Data used is hypothetical but serves to exhibit the model's capability. See Table A1.

Figures A1-A5 contain the actual data used in this sample simulation run. Figure A1B shows the road network divided into links. Note that the entry links are numbered greater than 800. Figure A5 contains the data in the correct format for input to the UTCS program. Figures A2, A3, A4 show the program's output listing of the input data. The model lists the data in easily readable format for verification of proper input. Before any further analysis is undertaken by the program user, one should carefully make sure that desired input corresponds with actual input. Figure A6 depicts the results of this simulation run with the network as shown in Figure A1 and the data as listed in Table A1.

Further Information

Material for this Appendix was taken from Reference 23. The prospective user should consult these self-explanatory documents should any questions arise.

Table A1. Network Data

Volume Entering Network

X Street:	250 (EB)	450 (WB)
Y Street:	250 (EB)	275 (WB)
Main Street:	425 (NB)	450 (SB)
Truck Volume:	2%	

Signal Timing

Main Street:	32 sec Green	3 sec Amber	25 sec Red
X, Y Streets:	22 sec Green	3 sec Amber	35 sec Red

Offset along Main Street from X Street to Y Street: 15 seconds

APPENDIX B

APPENDIX B: DELAY DIFFERENCE-OF-OFFSET PROGRAM AND INPUT FORMAT*

A computer program has been written for estimating the delay experienced by traffic because of queuing at a traffic signal. This calculation takes into account the vehicles that turned onto the main street at an upstream intersection and accumulated at the downstream intersection during its red phase. The delay is calculated for each one-way link specified in the input data deck. By computing the delay associated with each possible offset in one second increments, the program's output will allow the user to select a desirable offset.

A two-way street may be specified as two one-way links and designated primary and secondary as determined by the user. The output, as seen in Figure B1, shows the offsets (PHI), delay-in-queue during one signal cycle (QSUM), the average delay per vehicle (DPV), and the average queue length (QAVE). These parameters are given for the primary link, secondary link (opposite direction of primary link), and the combination of the two. Also included in the programs output is the link input data.

Input Format

Each link's characteristics are completely specified on one punched card. In order to have the results of two opposing links listed together the cards must be ordered so that the secondary link card immediately follows the opposing primary link card. Any number of sets of links may be input to this program for processing. The input format is as follows (all data must be right-justified). Figure B2 augments the encoding explanation.

---

*From References 54 and 57.

INPUT

LINK	C	G1	G2	A	SAT	LOST	D	W	ST	LT	RT	VH	V	OPPLNK	
3	50	19	25	4	.420	4	828	2.0	437.	34.	0	283.	30	4	P
4	50	25	19	4	.420	4	800	2.0	310.	34.	108.	506.	30	3	S

OUTPUT

PRIMARY LINK				SECONDARY LINK				COMBINATION			
PHI	QSUM	DPV	QAVE	PHI	QSUM	DPV	QAVE	PHI	QSUM	DPV	QAVE
0	47.9	12.2	.96	0	92.3	13.1	1.85	0	140.2	12.8	2.80
1	44.0	11.2	.88	49	93.0	13.2	1.86	1	137.0	12.5	2.74
2	39.9	10.2	.80	48	93.7	13.3	1.87	2	133.6	12.2	2.67
3	35.8	9.1	.72	47	94.3	13.4	1.89	3	130.0	11.9	2.60
4	31.3	8.0	.63	46	94.8	13.5	1.90	4	126.1	11.5	2.52
5	26.7	6.8	.53	45	95.3	13.6	1.91	5	122.0	11.1	2.44
6	22.0	5.6	.44	44	93.7	13.3	1.87	6	115.7	10.6	2.31
7	17.0	4.3	.34	43	92.1	13.1	1.84	7	109.2	10.0	2.18
8	11.9	3.0	.24	42	90.6	12.9	1.81	8	102.4	9.3	2.05
9	6.8	1.7	.14	41	89.0	12.7	1.78	9	95.7	8.7	1.91
10	3.0	.8	.06	40	87.4	12.4	1.75	10	90.4	8.3	1.81
11	.8	.2	.02	39	85.8	12.2	1.72	11	86.6	7.9	1.73
12	0	0	0	38	84.2	12.0	1.68	12	84.2	7.7	1.68
13	0	0	0	37	82.7	11.8	1.65	13	82.7	7.5	1.65
14	0	0	0	36	81.1	11.5	1.62	14	81.1	7.4	1.62
15	.2	.1	.00	35	79.5	11.3	1.59	15	79.8	7.3	1.60
16	.6	.2	.01	34	78.1	11.1	1.56	16	78.7	7.2	1.57
17	1.3	.3	.03	33	76.7	10.9	1.53	17	77.9	7.1	1.56
18	2.3	.6	.05	32	75.3	10.7	1.51	18	77.6	7.1	1.55
19	3.6	.9	.07	31	74.0	10.5	1.48	19	77.6	7.1	1.55
20	5.1	1.3	.10	30	72.7	10.3	1.45	20	77.8	7.1	1.56
21	7.0	1.8	.14	29	71.5	10.2	1.43	21	78.5	7.2	1.57
22	9.1	2.3	.18	28	70.4	10.0	1.41	22	79.5	7.3	1.59
23	11.5	2.9	.23	27	69.3	9.9	1.39	23	80.8	7.4	1.62
24	14.2	3.6	.28	26	68.3	9.7	1.37	24	82.5	7.5	1.65
25	17.2	4.4	.34	25	67.4	9.6	1.35	25	84.5	7.7	1.69
26	20.4	5.2	.41	24	66.5	9.5	1.33	26	88.9	7.9	1.74
27	24.0	6.1	.48	23	67.6	9.6	1.35	27	91.6	8.4	1.83
28	27.8	7.1	.56	22	68.8	9.8	1.38	28	95.6	8.8	1.93
29	31.8	8.1	.64	21	69.9	9.9	1.40	29	101.8	9.3	2.04
30	36.3	9.2	.73	20	71.1	10.1	1.42	30	107.4	9.8	2.15
31	41.0	10.4	.82	19	72.2	10.3	1.44	31	113.2	10.3	2.26
32	45.8	11.7	.92	18	73.4	10.4	1.47	32	119.2	10.9	2.38

Figure B1. Sample Program Output (Partial)

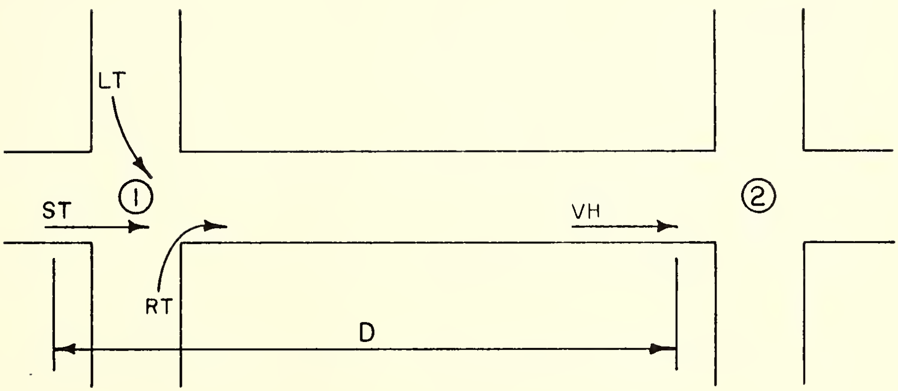

FIGURE B2: LINK REPRESENTATION

Cols.	Data Word
1-3	Link identification.
4-6	Cycle length (seconds).
7,8	Upstream green (seconds).
9,10	Downstream green (seconds.
11	Amber (seconds).
12-15	Single lane saturation flow (vehicles per second) in the form (.XXX). This is the average rate at which a single-lane queue discharges through intersection 2 during the effective green portion of the signal cycle.
16	Lost time per green interval at the link head, intersection 2, in seconds. Lost time results from starting delay at beginning of green and the subsidence of traffic flow during the yellow interval.
17,20	Link length (D) in feet (stop line to stop line).
21-23	Number of downstream lanes in the form (X.X). A fractional lane may be specified so as to represent small turn lanes that do not handle normal through volumes.
24-28	Upstream through traffic (ST) in vehicles per hour, of the form (XXXX.).
29-33	Upstream left turning traffic (LT) in vehicles per hour, of the form (XXXX.).
34-38	Upstream right turning traffic (RT) in vehicles per hour, of the form (XXXX.).
39-43	Downstream total traffic (VH)* in vehicles per hour of the form (XXXX.).

*VH generally does not equal ST+LT+RT because vehicles may be gained or lost between signalized intersections.

Cols.	Data Word
44,45	Average speed in miles per hour.
46-48	Opposite direction link identification.
49	P or S denoting primary or secondary link.

Program Listing

Figure B3 displays the delay difference-of-offset program listing which can be easily compiled and run on computers having FORTRAN IV compilers. This program is a refinement of the original version which only had provisions for analyzing one-way links separately with no regard for the opposing link. Its output provides a useful tool in selecting offsets for bettering traffic flow along an arterial.

```
      PROGRAM MAIN (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C     DELAY-DIFFERENCE OF OFFSET COMPUTATIONS,UNIDIRECTIONAL
C     LINK,TWO UNIFORM ARRIVAL RATES
C
C
C     FOR DATA INPUT FORMAT REFER TO NCHRP 124 PP 57-59.
C
C
      DIMENSION ITAU(151,3),IPHI(151,3),QSUM(151,3),Q(301),DPV(151,3),
     1QAVE(151,3),AVH(3)
      REAL LINK
      DATA BLNK,S,P/1H ,1HS,1HP/
C     WRITE HEADING FOR INPUT DATA
110   K=1
      WRITE(6,102)
103   FORMAT(1H0, 68HLINK C  G1 G2 A SAT  LOST  D   W    ST    LT    RT
     1   VH   V  OPPLNK)
      WRITE(6,103)
102   FORMAT(1H1, 5HINPUT)
C     READ INPUT DATA
100   READ(5,101)LINK,IC,IG1,IG2,IA,SAT,LOST,ID,W,AST,ALT,ART,AVH(K),IV,
     1OPPLNK,XLKCD
101   FORMAT(A3, I3,I2,I2,I1,F4.3,I1,I4,F3.1,F5.0,F5.0,F5.0,F5.0,I2,A3,
     1A1)
      IF(IC.EQ.0) GO TO 999
C     WRITE INPUT DATA
      WRITE(6,104)LINK,IC,IG1,IG2,IA,SAT,LOST,ID,W,AST,ALT,ART,AVH(K),
     1IV,OPPLNK,XLKCD
104   FORMAT (1H ,A3,1X,I3,1X,I2,1X,I2,1X,I1,1X,F4.3,2X,I1,2X,I4,1X,F3.1
     1,1X,F5.0,1X,F5.0,1X,F5.0,1X,F5.0,1X,I2,3X,A6,A5)
C     PERFORM INITIAL COMPUTATIONS
200   IR1=IC-IG1-IA
      IR2=IC-IG2-IA
      IT1=IG1+IA
      T1=IT1
      IT2=IR1
      T2=IT2
      IGE=IG2+IA-LOST
      IRE=IC-IGE
      GE=IGE
      AVT=AST+ALT+ART
      ANC=AVH(K)-AVT
      C=IC
      Q1=AST/((T1/C)*3600.)+ANC/3600.
      Q2=(ALT+ART)/((T2/C)*3600.)+ANC/3600.
      S=W*SAT
      V=IV
      V=(V*88./60.)+.5
      IV=V
C     CHECK SUBSATURATION
300   IF(S*GE-(Q1*T1+Q2*T2))301,400,400
C     DUMBHEAD OUTPUT
301   WRITE(6,302)
302   FORMAT(1H0, 31HDUMBHEAD,THIS IS SUPERSATURATED)
      GO TO 900
C     SET INITIAL TAU
400   IPHI(1,K)=0
C     SET UP OUTER LOOP TO COMPUTE QSUM FOR ALL TAUS
      DO 706 I=1,IC
C     COMPUTE TAU MODULO C
401   ITAU(I,K)=IPHI(I,K)-ID/IV-IR2
402   IF(ITAU(I,K)) 403,404,404
403   ITAU(I,K) = IC + ITAU(I,K)
      GO TO 402
C     SET INITIAL J AND QSUM
404   J=ITAU(I,K)
```

Figure B3. Source Listing of Delay-Difference Program

```
      QSUM(I,K)=0.
C     SET INNER LOOP TO COMPUTE QSUM FOR GIVEN TAU
405   ISTA=ITAU(I,K)+1
      IFIN=ITAU(I,K)+IC
406   DO 705 J=ISTA,IFIN
C     IS EFFECTIVE GREEN OR EFFECTIVE RED ON
      IF(J-(ITAU(I,K)+IRE))500,500,600
C     IS T1 OR T2 ON (EFFECTIVE RED)
500   IF(J-IT1)510,510,501
501   IF(J-IC) 520,520,502
502   IF(J-(IC+IT1))510,510,520
510   QCR=Q1
      GO TO 700
520   QCR=Q2
      GO TO 700
C     IS T1 OR T2 ON (EFFECTIVE GREEN)
600   IF(J-IT1)610,610,601
601   IF(J-IC)620,620,602
602   IF(J-(IC+IT1))610,610,620
610   QCR=Q1-S
      GO TO 700
620   QCR=Q2-S
C     COMPUTE Q AND ACCUMULATE QSUM
700   IF(J-(ITAU(I,K)+1))701,701,702
701   Q(J)=QCR
      GO TO 703
702   Q(J)=Q(J-1)+QCR
703   IF(Q(J))704,705,705
704   Q(J)=0.
705   QSUM(I,K)=QSUM(I,K)+Q(J)
C     END OF INNER LOOP
      QAVE(I,K) = QSUM(I,K)/C
      DPV(I,K) = QAVE(I,K)*3600.
706   IPHI(I+1,K) = IPHI(I,K) + 1
C     END OF OUTER LOOP
      K = K + 1
      IF (K.GT.2    .OR.OPPLNK.EQ.BLNK) GO TO 800
      GO TO 100
C     WRITE HEADING FOR OUTPUT DATA
800   WRITE (6,105)
105   FORMAT (1H0, 6HOUTPUT)
      WRITE(6,106)
      WRITE (6,107)
      IPAGE = 1
C     WRITE OUTPUT DATA
      DO 802 I = 1,IC
      IPAGE = IPAGE +1
      IF (IPAGE.LE.50) GO TO 422
      IPAGE = 1
      WRITE (6,21)
21    FORMAT (1H1)
      WRITE (6,106)
106   FORMAT (1H0,12HPRIMARY LINK,29X,14HSECONDARY LINK,27X,11HCOMBINATI
     1ON)
      WRITE (6,107)
107   FORMAT (1H0,3(28HPHI    QSUM     DPV     QAVE,13X))
422   JJ = IC - I + 2
      IF (JJ.GT.IC)  JJ = JJ - IC
      IF (OPPLNK.NE.BLNK)  GO TO 423
      QAVE (JJ,2) = 0.
      QSUM (JJ,2) = 0.
      DPV (JJ,2) = 0.
423   DPV(I,3) = (DPV(I,1) + DPV(JJ,2))/(AVH(1)+AVH(2))
      DPV(I,1) = DPV(I,1)/AVH(1)
      DPV(JJ,2) = DPV(JJ,2)/AVH(2)
      IPHI(I,3) = IPHI(I,1)
```

Figure B3. Continued

```
      QSUM (I,3) = QSUM(I,1) + QSUM(JJ,2)
      QAVE(I,3) = QAVE(I,1) + QAVE(JJ,2)
      WRITE (6,801) IPHI(I,1),QSUM(I,1),DPV(I,1),QAVE(I,1),IPHI(JJ,2),
     1QSUM(JJ,2),DPV(JJ,2),QAVE(JJ,2),IPHI(I,3),QSUM(I,3),DPV(I,3),
     2QAVE(I,3)
801   FORMAT (1H ,3(I3,F9.1,F8.1,F8.2,13X))
802   CONTINUE
C     IS THERE MORE DATA ≤ ≤
900   GO TO 110
999   CONTINUE
      WRITE(6,877)
877   FORMAT(1H0,10X,29H**** LAST LINK PROCESSED ****)
      STOP
      END
```

Figure B3. Continued

APPENDIX C

APPENDIX C: STREET REPRESENTATION AND VOLUME DATA

As further demonstration of the encoding process of a street system into a link and node configuration and to identify specific streets with their corresponding links, the link and node representations are illustrated. Figures C1, C2, and C3 show Northwestern Avenue, State Street, and Grant Street respectively. Also, so that signal timings may be further verified with some future volumes, the present-day volumes entering each of the three streets are listed in Tables C1, C2, and C3.

Referring to Figure C1, the signalized intersections are depicted by nodes 1, 10, 20, 30, and 40. Figure C2 which represents State Street has as its signalized intersections nodes 1, 2, 3, 4, 5, and 6. The signalized intersections along Grant Street (Figure C3) are nodes 1, 10, 50, and 60.

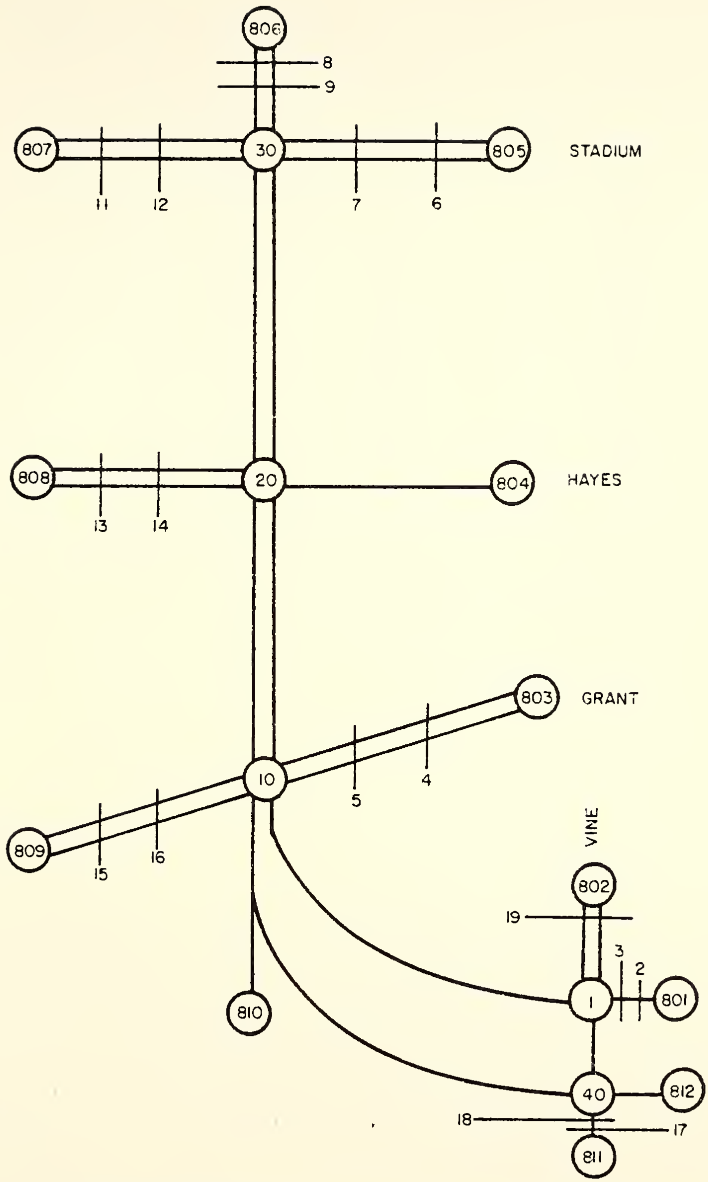

FIGURE C1: NORTHWESTERN AVENUE - LINK AND NODE REPRESENTATION

Table C1. Northwestern Avenue Volumes

Link	AM Peak	Off Peak	PM Peak
(801, 2)	732	414	443
(802, 19)	15	14	23
(803, 4)	360	182	297
(805, 6)	214	134	221
(806, 8)	699	430	483
(807, 11)	205	240	603
(808, 13)	109	154	272
(809, 15)	206	187	575
(811, 17)	234	224	286

Note: Volumes are in vehicles per hour.

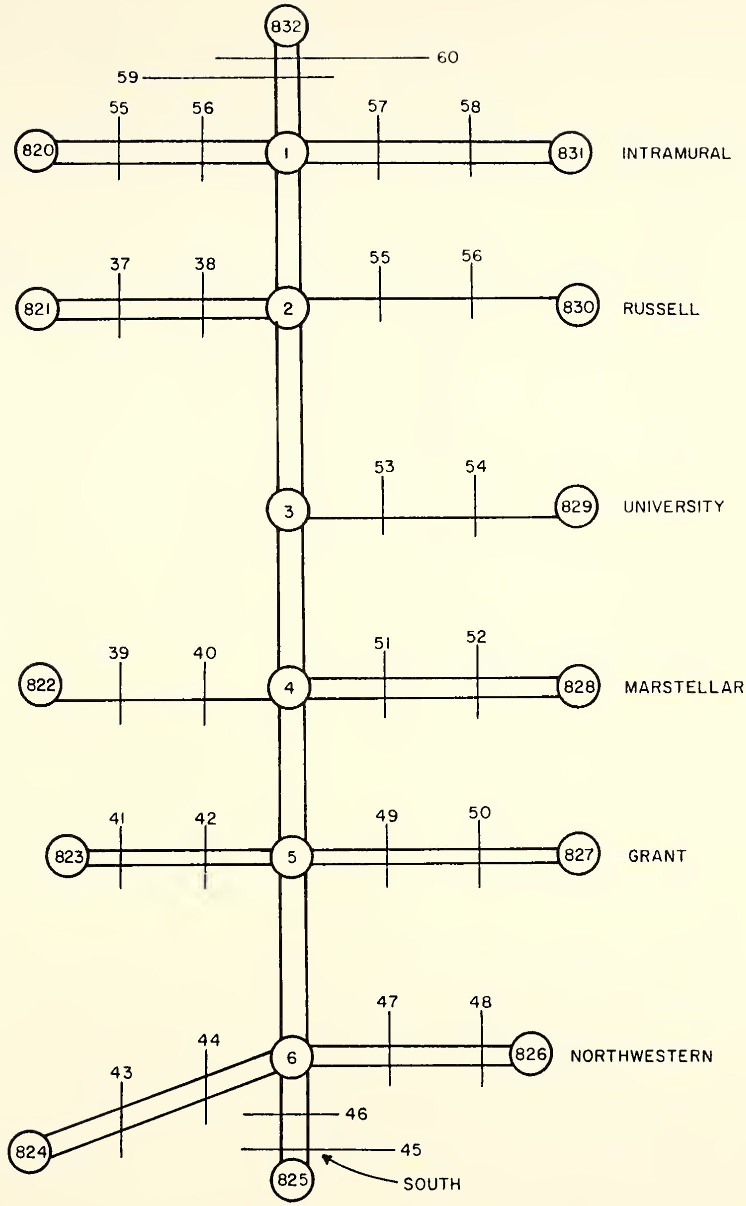

FIGURE C2: STATE STREET - LINK AND NODE REPRESENTATION

Table C2. State Street Volumes

Link	AM Peak	Off Peak	PM Peak
(820, 35)	71	62	146
(821, 37)	95	95	184
(822, 39)	130	178	343
(823, 41)	141	208	535
(824, 43)	617	549	538
(825, 45)	114	112	75
(826, 48)	204	353	354
(827, 50)	479	287	474
(828, 52)	51	36	119
(830, 56)	335	196	382
(831, 58)	47	89	127
(832, 60)	430	361	468

Note: Volumes are in vehicles per hour.

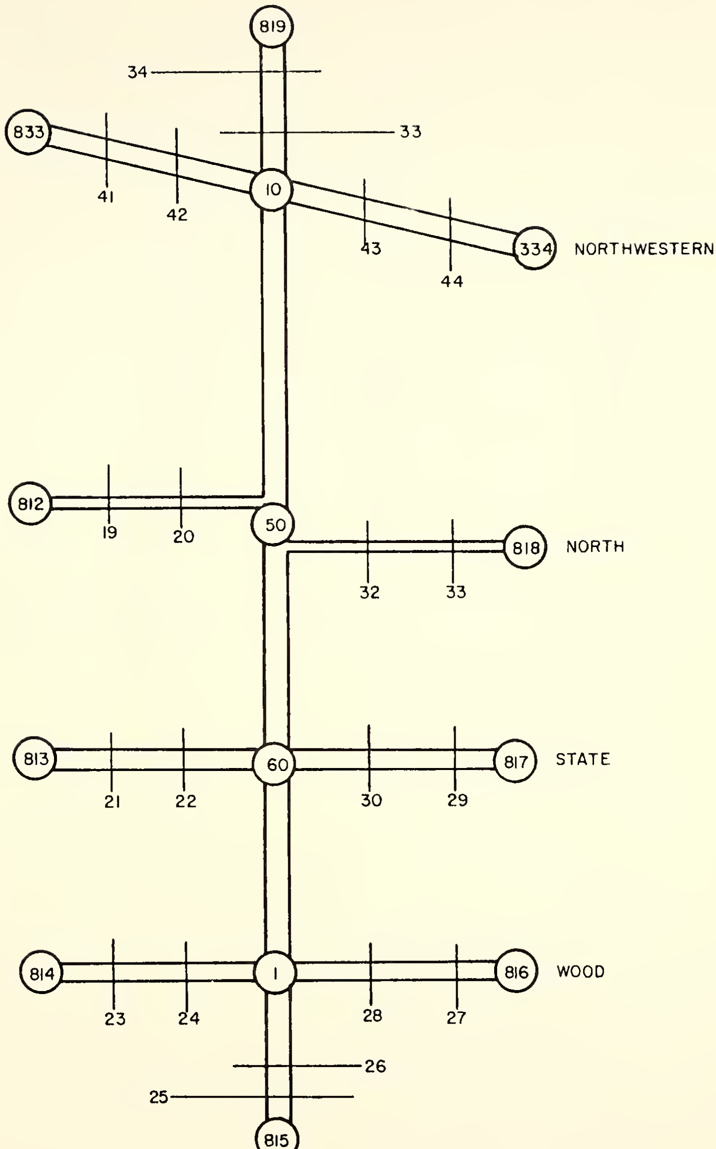

FIGURE C3 GRANT STREET - LINE AND NODE REPRESENTATION

Table C3. Grant Street Volumes

Link	AM Peak	Off Peak	PM Peak
(812, 19)	62	62	236
(813, 21)	458	562	1125
(814, 23)	195	123	523
(815, 25)	46	43	170
(816, 27)	392	100	253
(817, 29)	554	393	448
(818, 31)	168	105	257
(834, 44)	687	406	525
(819, 34)	360	182	297
(833, 41)	517	395	738

Note: Volumes are in vehicles per hour.

APPENDIX D

APPENDIX D: SIGNAL SETTINGS

The signal settings that were hereinbefore generated through the applied techniques are presented in this appendix. Also, the existing signal settings used as the base case for comparisons are listed. To condense the format with which these settings are shown, the following tables are modeled after the UTCS output.

Links and nodes are used to identify the approach that a particular phase is controlling. Reference is made to Appendix C Figures C1, C2, and C3 in determining the street approach from the link identification. To simplify tabular presentation further, signal codes (also, output from UTCS) are used to identify the specific phase indication. Table D1 lists the signal codes and their respective interpretation.

The tables listing the signal settings are organized as follows. First, the existing signal settings for Northwestern Avenue (Table D2) are shown. Following these are the developed Webster splits and cycles (Table D3). The offset plans (1 through 5) are then listed in Table D4. State Street (Tables D5-D7) and Grant Street (Tables D8, D9) signal settings are recorded in a similar fashion.

Table D1. Interpretation of Signal Codes

0	Yield or Amber
1	Green
2	Red
3	Red with Green Right Arrow
4	Red with Green Left Arrow
5	Stop or Red with Right Turn Permitted
6	Red with Green Diagonal Arrow
7	No Turns - Green Thru Arrow
8	Red with Left and Right Green Arrows
9	No Left Turn - Green Thru and Right

Table D2. Northwestern Avenue - Existing Signal Settings

		Duration (seconds)						
Node	Interval	AM	Off	PM	Signal Codes Facing Approach			
					(3,1)	(40,1)	(19,1)	
1	1	44	34	44	1	2	2	
1	2	5	4	5	0	2	2	
1	3	37	28	37	2	1	1	
1	4	4	4	4	2	0	0	
					(1,10)	(16,10)	(20,10)	(5,10)
10	1	49	37	49	1	5	1	2
10	2	5	4	5	0	5	0	2
10	3	31	25	31	5	1	5	1
10	4	5	4	5	5	0	5	0
					(10,20)	(14,20)	(30,20)	
20	1	45	27	45	1	5	1	
20	2	5	4	5	0	5	0	
20	3	22	22	22	5	1	5	
20	4	5	5	5	5	0	5	
20	5	13	12	13	4	5	4	
					(20,30)	(12,30)	(9,30)	(7,30)
30	1	18	14	19	1	5	5	5
30	2	36	29	25	1	5	1	5
30	3	5	4	5	0	5	0	5
30	4	27	18	37	5	1	5	1
30	5	4	5	4	5	0	5	0
					(21,40)	(18,40)		
40	1	44	34	44	1	2		
40	2	5	5	5	0	2		
40	3	37	26	37	2	1		
40	4	4	5	4	2	0		

Table D3. Northwestern Avenue - Webster Cycles and Splits

		Duration (secs)						
Node	Interval	AM	Off	PM	Signal Codes Facing Approach			
					(3,1)	(40,1)	(19,1)	
1	1	40	27	33	1	2	2	
1	2	4	4	4	0	2	2	
1	3	22	15	29	2	1	1	
1	4	4	4	4	2	0	0	
					(1,10)	(16,10)	(20,10)	(5,10)
10	1	38	27	36	1	5	1	2
10	2	4	4	4	0	5	0	2
10	3	24	15	26	5	1	5	1
10	4	4	4	4	5	0	5	0
					(10,20)	(14,20)	(30,20)	
20	1	37	19	30	1	5	1	
20	2	4	3	4	0	5	0	
20	3	15	15	22	5	1	5	
20	4	4	3	4	5	0	5	
20	5	10	10	10	4	5	4	
					(20,30)	(12,30)	(9,30)	(7,30)
30	1	11	7	8	1	5	5	5
30	2	31	18	22	1	5	1	5
30	3	4	4	4	0	5	0	5
30	4	20	17	32	5	1	5	1
30	5	4	4	4	5	0	5	0
					(21,40)	(18,40)		
40	1	35	23	43	1	2		
40	2	4	4	4	0	2		
40	3	27	19	19	2	1		
40	4	4	4	4	2	0		

Table D4. Northwestern Avenue Offsets

Timing Plan	Offsets (seconds) to: Node 1*	Node 10	Node 20	Node 30	Node 40
AM Peak:					
1	0	45	38	19	0
2	0	12	83	79	0
3	0	22	5	62	0
4	0	1	37	34	2
5	0	15	28	59	45
Off Peak:					
1	0	60	30	57	60
2	0	18	63	63	0
3	0	17	48	21	0
4	0	0	28	1	3
5	0	13	33	1	2
PM Peak:					
1	0	45	38	19	0
2	0	63	49	68	0
3	0	14	2	22	0
4	0	1	37	34	2
5	0	15	28	59	45

*Time reference point.

Note: All offsets are to Interval 1.

Table D5. State Street - Existing Signal Settings

Node	Interval	Duration (seconds)	Signal Codes Facing Approach			
			(59,1)	(57,1)	(2,1)	(36,1)
1	1	59	1	5	1	5
1	2	4	0	5	0	5
1	3	23	5	1	5	1
1	4	4	5	0	5	0
			(1,2)	(55,2)	(3,2)	(38,2)
2	1	50	1	5	1	5
2	2	4	0	5	0	5
2	3	32	5	1	2	1
2	4	4	5	0	2	0
			(2,3)	(4,3)		
3	1	59	1	1		
3	2	4	0	0		
3	3	27	4	2		
			(3,4)	(51,4)	(5,4)	(40,4)
4	1	18	1	2	5	5
4	2	41	1	2	1	5
4	3	4	0	2	0	5
4	4	23	2	1	5	1
4	5	4	2	0	5	0
			(4,5)	(49,5)	(6,5)	(42,5)
5	1	18	1	2	5	5
5	2	37	1	2	1	5
5	3	4	0	2	0	5
5	4	28	5	1	2	1
5	5	4	5	0	2	0

Table D5. Continued

Node	Interval	Duration (seconds)	Signal Codes Facing Approach			
			(5,6)	(47,6)	(46,6)	(44,6)
6	1	29	6	2	2	1
6	2	5	6	2	2	0
6	3	25	1	2	1	2
6	4	4	0	2	0	2
6	5	27	2	1	2	7

Note: AM, Off, and PM Peak timings are identical.

Table D6. State Street - Webster Cycles and Splits

		Duration (secs)						
Node	Interval	AM	Off	PM	Signal Codes Facing Approach			
					(59,1)	(57,1)	(2,1)	(36,1)
1	1	48	47	57	1	5	1	5
1	2	4	4	4	0	5	0	5
1	3	15	15	15	5	1	5	1
1	4	3	4	4	5	0	5	0
					(1,2)	(55,2)	(3,2)	(38,2)
2	1	38	42	46	1	5	1	5
2	2	4	4	4	0	5	0	5
2	3	24	20	26	5	1	2	1
2	4	4	4	4	5	0	2	0
					(2,3)	(4,3)		
3	1	42	48	58	1	1		
3	2	4	4	4	0	0		
3	3	24	18	18	4	2		
					(3,4)	(51,4)	(5,4)	(40,4)
4	1	10	10	10	1	2	5	5
4	2	35	32	42	1	2	1	5
4	3	4	4	4	0	2	0	5
4	4	18	20	20	2	1	5	1
4	5	3	4	4	2	0	5	0
					(4,5)	(49,5)	(6,5)	(42,5)
5	1	10	13	16	1	2	5	5
5	2	29	23	30	1	2	1	5
5	3	4	4	4	0	2	0	5
5	4	23	26	25	5	1	2	1
5	5	4	4	4	5	0	2	0

Table D6. Continued

		Duration (secs)			Signal Codes Facing Approach:			
Node	Interval	AM	Off	PM	(5,6)	(47,6)	(46,6)	(44,6)
6	1	30	28	32	6	2	2	1
6	2	4	4	4	6	2	2	0
6	3	15	17	23	1	2	1	2
6	4	3	3	3	0	2	0	2
6	5	18	18	18	2	1	2	7

Table D7. State Street Offsets

Timing Plan	Offsets (seconds) to: Node 1*	Node 2	Node 3	Node 4	Node 5	Node 6
AM Peak:						
1	0	0	42	27	85	7
2	0	24	0	0	73	66
3	0	26	7	66	51	35
4	0	4	37	36	8	7
5	0	14	29	25	0	0
Off Peak:						
1	0	0	42	27	85	7
2	0	83	63	63	46	17
3	0	59	41	63	32	61
4	0	2	34	38	2	9
5	0	8	2	24	0	0
PM Peak:						
1	0	0	42	27	85	7
2	0	83	61	76	59	8
3	0	75	54	72	63	10
4	0	5	41	43	4	12
5	0	16	34	32	0	0

*Time reference point.

Note: All offsets are to Interval 1.

Table D8. Grant Street - Existing Signal Settings

Node	Interval	Duration (secs) AM	Off	PM	Signal Codes Facing Approach			
					(60,1)	(28,1)	(26,1)	(24,1)
1	1	8	8	8	1	2	2	5
1	2	18	18	18	1	2	1	5
1	3	4	4	4	0	2	0	5
1	4	8	8	8	5	2	2	1
1	5	32	32	32	5	1	2	1
					(33,10)	(43,10)	(50,10)	(42,10)
10	1	31	25	31	1	5	1	5
10	2	5	4	5	0	5	0	5
10	3	49	37	49	2	1	5	1
10	4	5	4	5	2	0	5	0
					(10,50)	(32,50)	(60,50)	(20,50)
50	1	39	39	39	1	5	1	2
50	2	4	4	4	0	5	0	2
50	3	6	6	6	5	5	5	2
50	4	18	18	18	5	1	5	1
50	5	3	3	3	5	0	5	0
					(50,60)	(30,60)	(1,60)	(22,60)
60	1	28	28	28	1	2	1	2
60	2	4	4	4	0	2	0	2
60	3	18	18	18	2	2	5	1
60	4	36	36	36	2	1	5	1
60	5	4	4	4	2	0	5	0

Table D9. Grant Street - Webster Cycles and Splits

Node	Interval	Duration (secs) AM	Off	PM	Signal Codes Facing Approach			
					(60,1)	(28,1)	(26,1)	(24,1)
1	1	8	6	8	1	2	2	5
1	2	18	15	26	1	2	1	5
1	3	4	4	4	0	2	0	5
1	4	8	6	8	5	2	2	1
1	5	22	19	24	5	1	2	1
					(33,10)	(43,10)	(50,10)	(42,10)
10	1	24	15	26	1	5	1	5
10	2	4	4	4	0	5	0	5
10	3	38	27	36	2	1	5	1
10	4	4	4	4	2	0	5	0
					(10,50)	(32,50)	(60,50)	(20,50)
50	1	32	24	33	1	5	1	2
50	2	4	4	4	0	5	0	2
50	3	5	5	5	5	5	5	2
50	4	15	14	24	5	1	5	1
50	5	4	3	4	5	0	5	0
					(50,60)	(30,60)	(1,60)	(22,60)
60	1	23	26	25	1	2	1	2
60	2	4	4	4	0	2	0	2
60	3	10	13	16	2	2	5	1
60	4	29	23	30	2	1	5	1
60	5	4	4	4	2	0	5	0

COVER DESIGN BY ALDO GIORGINI